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8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

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ARCH DAMS: PORTUGUESE EXPERIENCE WITH OVERFLOW ARCH DAMS*

A. C. Xerez**
(Proc. Paper 990)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds: "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Gustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 990 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

*Paper presented at the Symposium on Arch Dams of the ASCE, Power Div., June, 1956.

**Technical Director, Hidro-Eléctrica do Zezere (Portugal), Pres., Civ. Eng. Section of the "Ordem dos Engenheiros" (Portugal), Pres., Hydroelectric Div. of the "Ordem dos Engenheiros."

SYNOPSIS

An important part of every dam, affecting both safety and cost, is the arrangement by which flood waters by-pass the dam. This paper discusses Portuguese experience with arch dam spillways. Discussed are the problems involved in the various types and presented are actual examples.

The delicate problem of deciding upon the layout of the appurtenant works of a dam is intimately related to that of flood discharge. The problem arises as soon as it becomes necessary to estimate the maximum flood and to establish its hydrograph. The difficulties in obtaining these elements are very well known to hydraulic engineers who appreciate the responsibilities involved as regards the safety of the dam.

The examination of this safety should be faced not only from the point of view of strength of the structure; it must be taken further, bearing in mind, particularly, the question of flood discharge. It frequently happens that engineers who are not fully aware of the problem of flood discharge are led to arrive at solutions which they feel to be the most convenient when, should the problem be studied in all its extent and due consideration paid to the hydraulic problem, their conclusions might be very different. These considerations are related to the more general problem of the choice of the most convenient type of dam for the particular site conditions, having in mind both its safety and its cost. This problem was dealt with at the last Congress on Large Dams at which the conclusion was reached that the choice is always dependent on many factors which require individual analysis by the designer in each case.

In this connection it should be stressed that one of the most important factors is that of flood discharge which can introduce determining factors of such a nature as to alter completely the type of dam which would otherwise have been chosen. Thus it can happen that, for a certain type of valley, where a thin arch dam could be built to advantage, a solution has to be chosen involving the building of other type of dam. This will be the case if the floods to be discharged are so large that this solution would give the best overall technical and economical results when the dam and flood discharge arrangements are considered together as one whole. It appears that American engineers are well aware of this aspect of the problem because of the large floods which must be foreseen in the case of their own schemes. The reasons will therefore be well appreciated for drawing attention to this matter here.

From what has been written above, i.e., the important determining factors introduced by flood discharge, the preference which designers may have for arch dams because of their inherent reserve of strength, must be considered in conjunction with this problem particularly when large floods have to be handled. Therefore it seems justified that designers call upon the cooperation of hydraulic engineers in order to arrive at solutions which will permit of the maximum advantage being taken from the arched form. This will be possible when an arch dam discharging over the crest can be built because in such cases can the dam and flood discharge together show economic advantages over the gravity dam.

Another factor arises if the dam forms part of a hydroelectric scheme where the power station is located at its foot as this will influence the layout of the accessory installations as well as the choice of the best type of dam.

General Considerations Concerning Overflow Arch Dams

The problem of flood discharge over arch dams can be tackled by the designer in various aspects, all depending on the flow to be discharged, the height of the dam and its type. The solution to such problems varies from the thick arch dam with overfall spillway to the thin arch dam with free flood discharge.

If the flood is small no technical difficulties arise in designing overflow arch dams even in the case of high dams, and, from an economical point of view, this solution is clearly the best.

If the flood is large and the valley has topographical and geological characteristics which warrant an arch dam, then the choice of a solution for the best dam and flood discharge taken together as a unit—bearing also in mind other determining factors such as, for instance, a power station located at the foot of the dam—requires very careful study, case by case.

Where the power station is not located up against the dam itself and the valley is suitable for a thin arch dam the best technical and economical solution consists in freely discharging the flood over all or part of the crest, controlled by flood gates particularly in the latter case, providing the flow per unit length of crest is not excessive and the dam is of medium height.

When the arch dam is of the thick type, advantage can be taken of the slope of its downstream face for flood discharge although this will require the nappe to be forced to follow its shape. In this case also, if the width of the valley permits, it will be possible to locate the power station at the foot of the dam close to the spillway.

When the flow per unit length of crest is relatively high, over 80 m3/sec. per meter of crest (860 cu. ft./sec. per foot of crest), or the height of the dam exceeds 100 metres (328 ft.), the problem must be very carefully studied, specially in the case of free discharge where it becomes very delicate even for lower values than these. In such cases, although a solution involving flood discharge over the crest of an arch dam would be the most favourable from an economic point of view, solutions requiring independent spillways must also be considered. The possibility of an overflow gravity dam should also be studied as, hydraulically, it may offer greater safety.

The Hydraulic Problem and Model Tests

As far as the design of the crest is concerned, the hydraulic problem of flood discharge over a dam may be easily solved by analytical methods if the discharge is free. It cannot however be so readily solved by analysis in cases where the nappe has to be forced to follow the shape of the downstream face of the dam and difficulties become even greater if the water approaches the spillway asymmetrically.

The remaining problems requiring attention whether overfall be free or

whether the nappe follows the shape of the downstream face including that very important one of energy dissipation downstream, are not open to solution by analysis and, as generally accepted, involve exhaustive model tests.

One conclusion to be drawn from the above is that the hydraulic model is an indispensable tool to the designer of dams when studying the various possible solutions; firstly using reduced scale models for preliminary studies and, later, larger models for the final and complete tests on the chosen solution. This procedure is fundamental for arriving at the best design. It is the writer's opinion that the method sometimes adopted, where the designer carries out his complete study in his drawing office and effects model tests only on the solution of his choice arrived at purely by analysis, without even directing or assisting at these model tests, is not justified.

The problem of downstream energy dissipation is the most difficult one connected with hydraulic tests. In many solutions involving overfall discharge, even in cases where the nappe follows the shape of the downstream face of the thick arch dams, the jet can be thrown on the bed of the river and hydraulic jump will not occur. The problem to be faced, therefore, is that of dissipating the energy of a jet of water falling through a given height as determined by the difference on elevation between the upstream and downstream water levels. This is the reason for the remarks made above to the effect that difficulties in arriving at safe solutions to the hydraulic problem increase as the flow per unit length of crest and the height of the dam increase. However, on the other hand, the narrower the valley, the greater will be the depth of the water in the river bed for higher discharges and this will facilitate energy dissipation.

This energy dissipation is the cause of erosion of the river bed and banks, mainly as a result of the well known phenomenon of uplift developed through cracks and diaclases in the rocks. Due to the nature of this phenomenon and as it is impossible to reproduce the actual nature of the rocks in the model, hydraulic tests cannot show the real results and, therefore, the problem must be approached by assuming the worst possible conditions so as to determine whether erosion is regressive in such a way as to cause damage to the dam foundations. In general, regressive erosion need not be feared and, therefore, the problem becomes one of analysing the pool formed in the bed of the river whose shape and depth will ultimately become stabilized. The resulting barrier across the river will have no adverse effect on the hydroelectric scheme except when formed downstream of the turbine tailrace, but in any case its removal presents no difficulties.

Structural Design Problem and Model Tests

If free discharge takes place over the whole length of the crest of a thin arch dam, no special structural design problems are involved. But, if the discharge is limited to one part of the crest, whether the dam be of the thin or thick arch type, then large intake openings, closed by gates, must be left in the arch. This brings about the problem of the re-establishment of the continuity of the arch effect through the reinforced concrete structure encasing these openings.

This problem may require the use of heavy reinforcement, not only for the purpose of ensuring structural continuity but also for supporting the gates and the resulting increase in cost must be taken into consideration when comparing this solution with others.

The study and design of these structures involve delicate problems of elasticity which are not easily approached by analysis unless simplifications be introduced in the computation. The technique of model tests, which has become so advanced throughout the world, is an indispensable working tool if used in conjunction with analytical studies but it must be borne in mind that it is up to the designer to conduct the studies in such a manner as to take full advantage of these two methods, i.e., model tests and analysis.

Some Recent Portuguese Overflow Arch Dams

Portuguese experience in the building of overflow arch dams is of interest because, having resulted from the work carried out in recent years, it has consequently benefitted from the experience gained in other countries. As a result of the national electrical development plan which is being carried out in Portugal, there are six hydroelectric schemes in operation where arch dams more than 60 metres (197 ft) high have been built. Five of these are overflow arch dams of various types.[10] Of the schemes now under construction, one of the dams is of the rock fill type 110 metres (361 ft) high and the other is a thin arch dam 90 m (295 ft) high. The latter is of the overflow type handling large floods per unit length of crest with special arrangements for the restitution of the water downstream of the dam.

This shows that in a large percentage of hydroelectric schemes, the type of overflow arch dam has been adopted. The reason for this was mainly one of economy, studies having shown that a satisfactory and safe solution to the hydraulic problem was possible. The chosen sites were suitable for the building of this type of dam and careful analytical studies and model tests had to be made with a view to arriving at the most convenient and safest solution in each case. The flood discharge over the six dams can be classified in distinct types according to their main characteristics and detailed reference is made to each below.

The design of the dam and of the spillway considered as a unit depends fundamentally on the flow to be discharged and on the layout of the accessory installations, including the power station when this is located near the dam.

The existence of a foot-of-dam type power station makes it extremely difficult to discharge floods over the dam unless an arrangement be adopted such as that which the French consulting engineer A. Coyne has used where the flood is discharged over the dam and power station together. This same engineer advised the adoption in Portugal of lateral overfall spillway in the case of the thick arch dams of Castelo do Bode and Venda Nova. The former dam has a height of 110 m (361 ft) with a foot-of-dam power station situated close to the spillway (Fig. 1).

This discharge over the downstream face of a thick arch dam is possible only within certain limits of slope and on condition that the nappe is forced to take the shape of this face by means of a especially designed intake opening

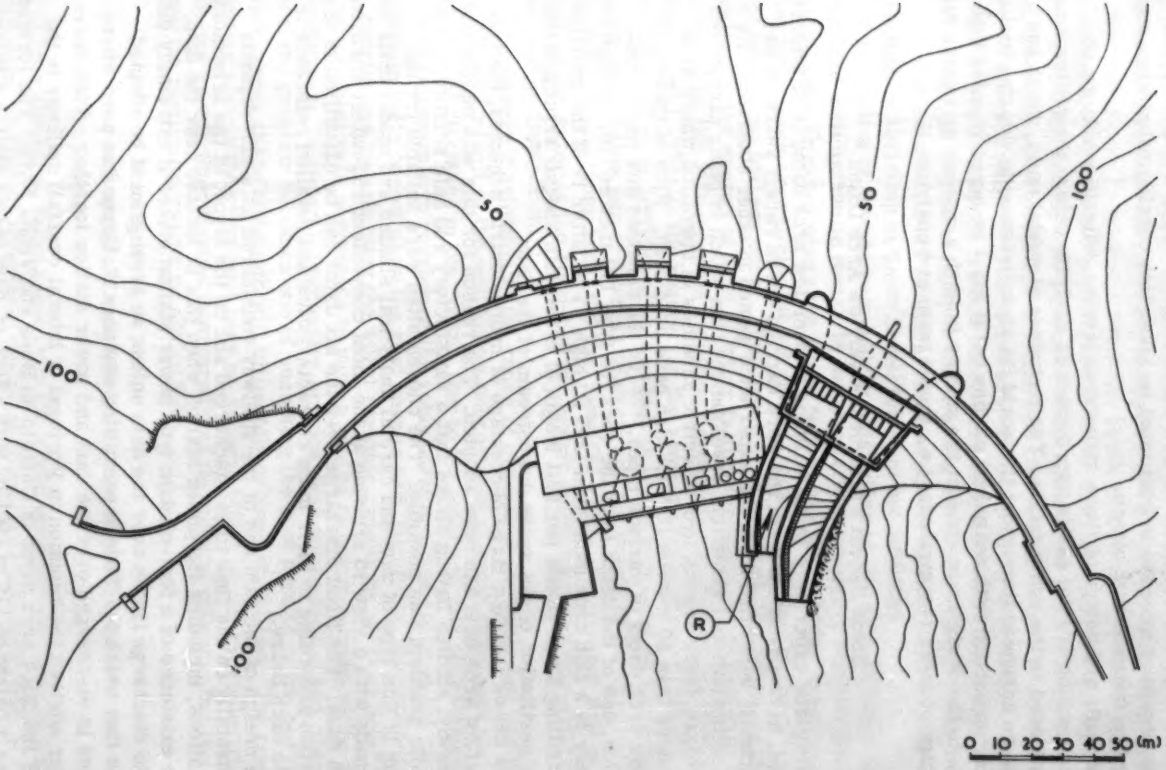

FIG. 1 — CASTELO DO BODE — PLAN OF THE DAM, POWER STATION AND SPILLWAY

fitted with control gates. In the case of Castelo do Bode, a solution was arrived at which will permit the discharge of 4,000 m3/sec (141,260 cu ft/sec through two openings each measuring 14.00 x 10.00 m (45.93 x 32.81 ft), fitted with radial gates (Fig. 2).

The restitution of the water downstream of the dam after leaving the spillway channels presented certain difficulties due to the limited space available between the power station and the left bank of the river and because of the necessity of ensuring that the jets would be directed along the axis of the river in such a way as to permit perfect aeration of the jets and ascertain that the banks would not be eroded. After exhaustive hydraulic tests, a satisfactory solution became possible by narrowing the jets; this permitted the width of the spillway channels to be restricted and allowed perfect energy dissipation in the air before the jets reached the river. This solution also made it easier to throw the jets in the desired direction. As regards the effect of the jets falling on the river bed, model tests showed that a natural pool is formed its shape becoming stabilised and that no regressive erosion takes place which might adversely affect the dam foundations. Actual experience during recent floods (Fig. 3) confirmed the conclusions arrived at the model tests.

Apart from the hydraulic problems mentioned above, the adoption of this solution required special analytical studies and model tests in relation to the reinforced concrete structure surrounding the discharge openings in the dam in order to maintain the continuity of the arch effect.

The above summarises the main aspects of one layout with a thick overflow arch dam having a power station located at its foot and which was the first one to be built in Portugal involving the discharge of large floods over a high dam.

Reverting to a layout with a power station situated close to the dam, the building of an overflow dam is simplified where it is possible to locate the power station laterally in one of the banks, leaving space enough between it and the dam to facilitate free flood discharge over its crest. This was the solution which was recently adopted in the case of the thin arch dam at Bouçã 60 m (197 ft) high (Figs. 4, 5, 6, 7), where 2,200 m3/sec (77,700 cu ft/sec) can be discharged over the whole crest with a nappe height of 3.5 m (11.48 ft), the water falling directly into a concrete lined basin. The crest is divided into three sections at two different levels; one central section and two lateral sections. The former is capable of discharging 300 m^3/sec (10,590 cu ft/sec) before the others come into operation so ensuring the formation of a stilling pool at the foot of the dam to facilitate energy dissipation.

The hydraulic model tests enabled the determination of the length of crest required for flood discharge, the correct level for the central and lateral sections and their best profil taking into account the most convenient area on which the jet should fall. These tests also permitted the study of the best method for dividing the nappe by means of piers situated on the crest in order to eliminate the well known vibratory effect which can be set up at the beginning of the discharge when the nappe is thin. By means of these tests it was further possible to design the shape of the basin to enable on one hand energy dissipation to take place satisfactorily and, on the other, to ensure

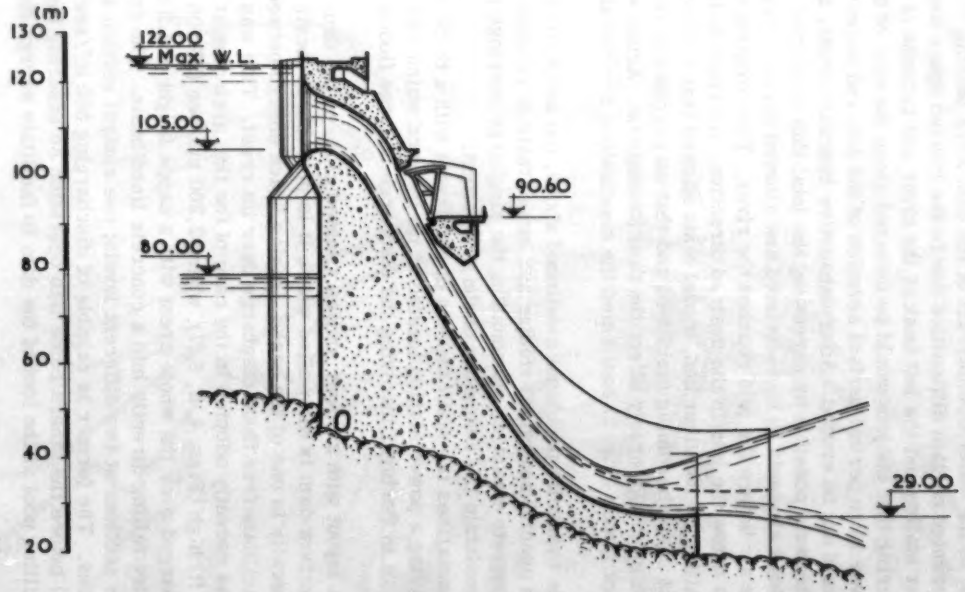

FIG. 2 – CASTELO DO BODE – SECTION THROUGH THE RIGHT SPILLWAY CHANNEL (R)

Fig. 3 - CASTELO DO BODE - Flood discharge of 1,100 m^3/sec. (38,850 cu.ft./sec.). Outlets of the channels, viewed from the crest of the dam.

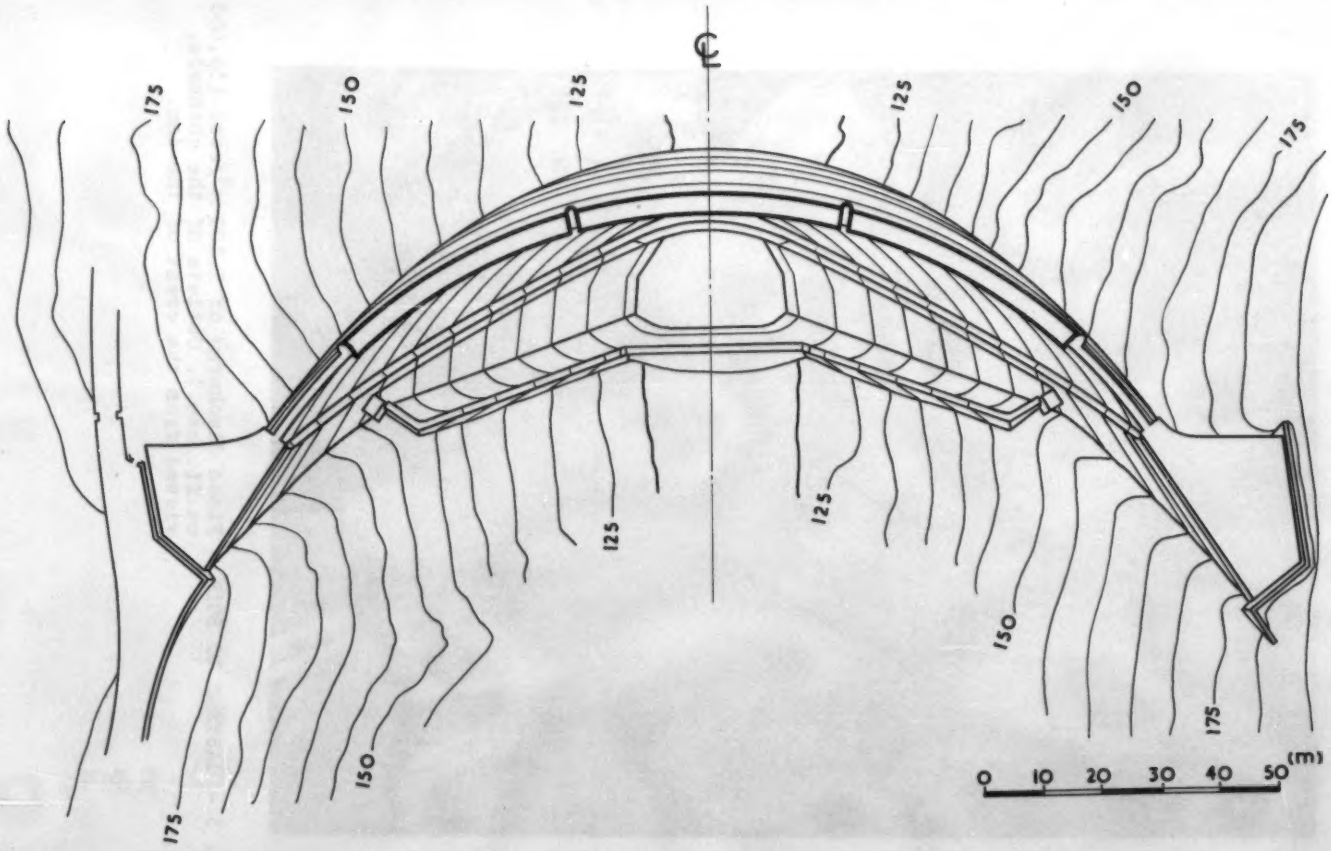

FIG. 4 — BOUÇÃ — PLAN OF THE DAM AND SPILLWAY

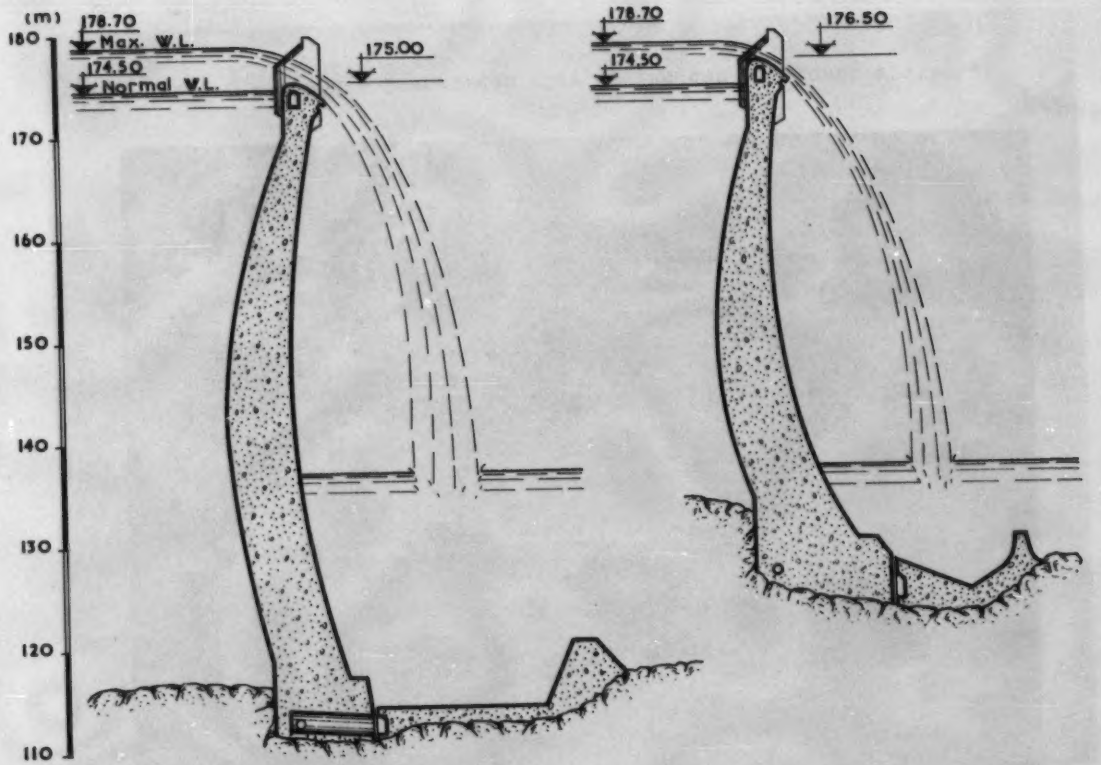

FIG. 5 — BOUÇÃ — CENTRAL AND LATERAL PROFILES OF THE DAM AND SPILLWAY

Fig. 6 - BOUÇÃ - Downstream view of the dam and power station.

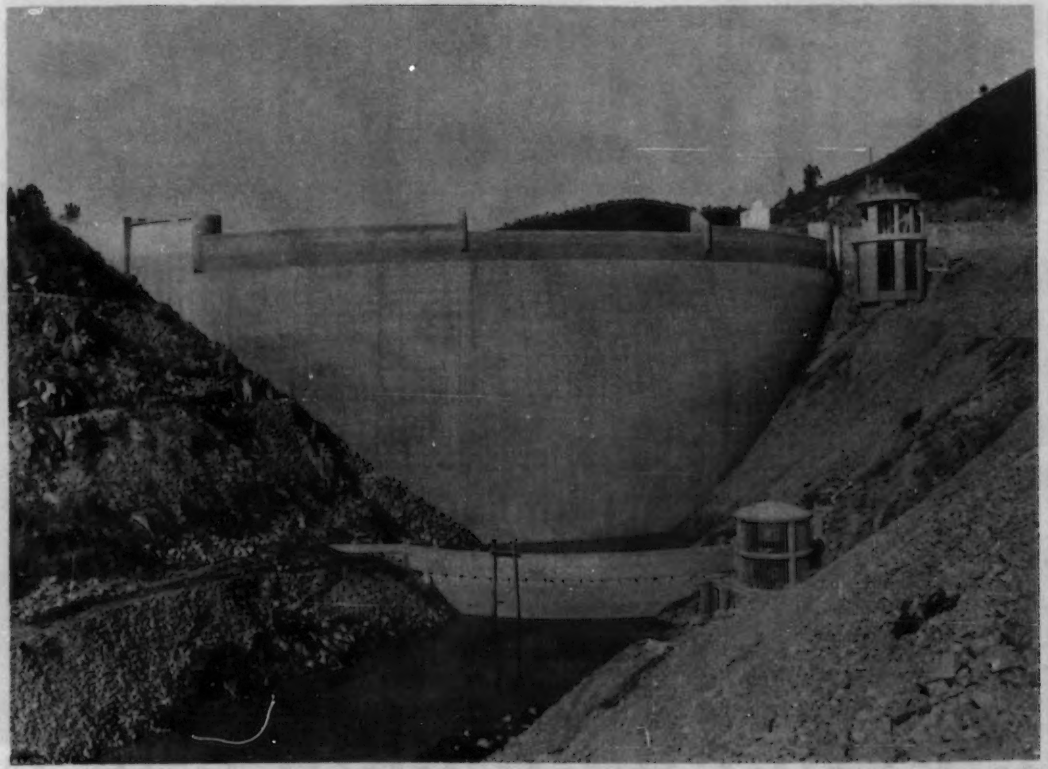

Fig. 7 - BOUÇÃ - Upstream view of the dam and intake structure.

that restitution downstream took place in such a way as to avoid undue disturbance in the vicinity of the power station under all flood conditions, mainly for the most frequently expected flow of about 500 m3/sec (17,660 cu ft/sec).

In last February, the river Zezere was in heavy flood and a maximum flow of 1,500 m3/sec (52,980 cu ft/sec) was experienced. During the discharge (Fig. 8) the absence of disturbance in the vicinity of the power station and the satisfactory conditions of energy dissipation were confirmed and coincided within the normal similarity limitations with the observations made during model tests (Fig. 9).

This type of flood discharge does neither affect the arch form nor does it bring about any special constructional problem except the one resulting from the convenience in giving geometric continuity to the downstream face and spillway basin and the establishment of an expansion joint between these two elements which have different elastic characteristics.

Flood discharge over a thin arch dam can also be restricted to its central portion using openings fitted with control gates, usually of the stoney type, to attend to space limitations. This third layout for the dam and spillway has been adopted at Salamonde and Caniçada (Figs. 10, 11) where the power stations are underground. The former has a height of 75 m (246 ft) and is capable of handling floods of 1,700 m3/sec (60,037 cu ft/sec), the corresponding values for the latter being approximately the same. This type of flood discharge does not demand special hydraulic tests as compared with the previous type nor are any special constructional problems involved downstream. The only care required is the consolidation and lining of a small area to receive the impact of the jet.

Free flood discharge over a thin arch dam offers a simple solution and a cheap construction can be used. However, when large floods per unit length of crest are discharged, this solution for dams of great height must be regarded with care from a safety point of view. This was the reason why this type of discharge was not adopted at Cabril[11] where the dam has a height of 135 metres (443 ft) and must handle floods of the order of 2,000 m3/sec (70,600 cu ft/sec). Although extensive tests were made on the model of an overflow dam, the solution of having independent flood discharge by means of two tunnels, one on each bank, was finally selected (Figs. 12, 13, 14).

Finally, mention will be made of the type of overflow arch dam arrangement most recently adopted in Portugal and which is now under construction. The problem put before the designers was the building of a dam about 90 m (295 ft) high in a gorge of well consolidated granite having a width, at the level of the crest, of little more than 100 m (328 ft) and where exceptional floods of 11,000 m3/sec (388,470 cu ft/sec) had to be contemplated. This is a case of having to deal with a large flow per unit length of crest where, due to the great height of the dam, a very large amount of energy has to be dissipated under flood conditions. The dam forms part of the hydroelectric project of Picote on the international section of the river Douro.

The difficulties in arriving at a satisfactory layout for the dam, spillway and power station can be appreciated from what has been written above bearing in mind the narrowness of the gorge and the convenience of taking full advantage of the geological and topographical conditions which are so favorable for the building of an arch dam. This is a typical case which conforms

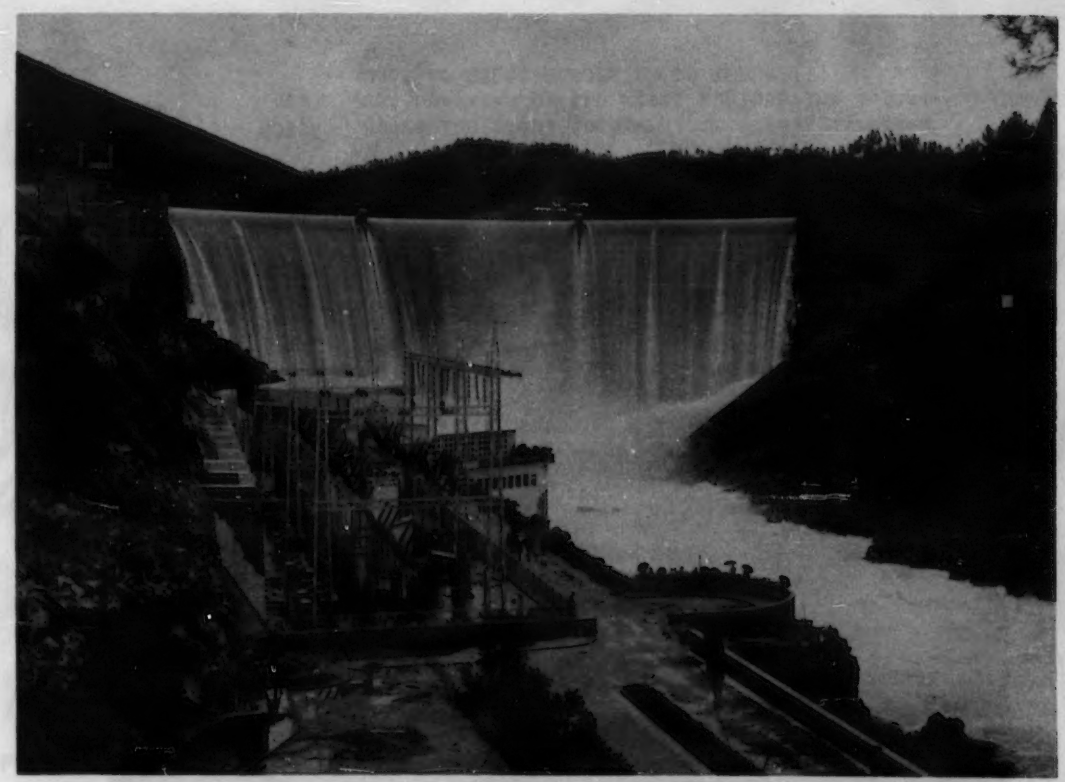

Fig. 8 - BOUÇÃ - Flood discharge of 1,300 cu.m./sec. (45,910 cu.ft/sec.)

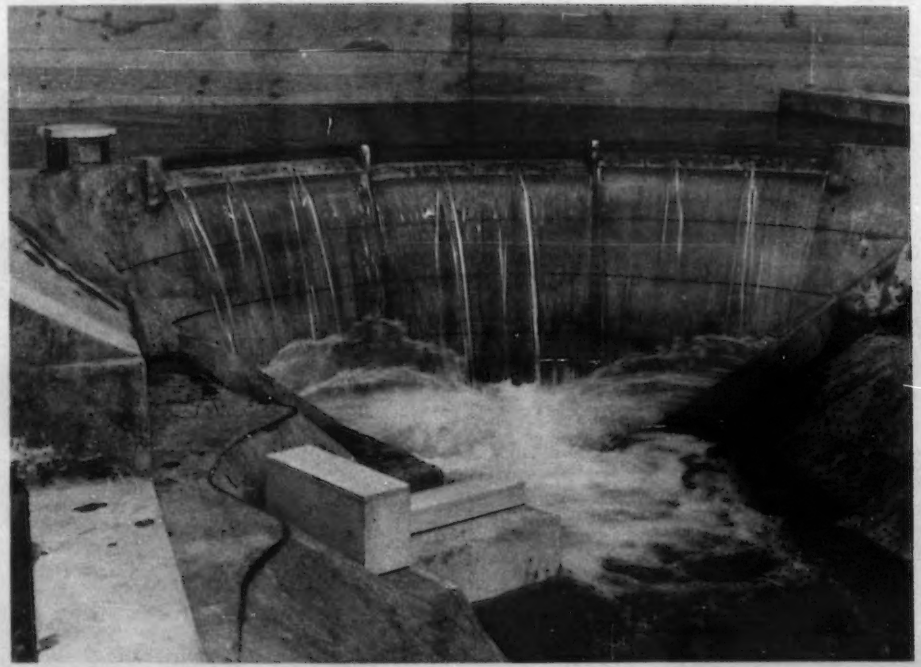

Fig. 9 - BOUÇÃ - Hydraulic model in operation (Tests directed and operated by the Civil Engineering Department of HIDRO-ELECTRICA DO ZEZERE).

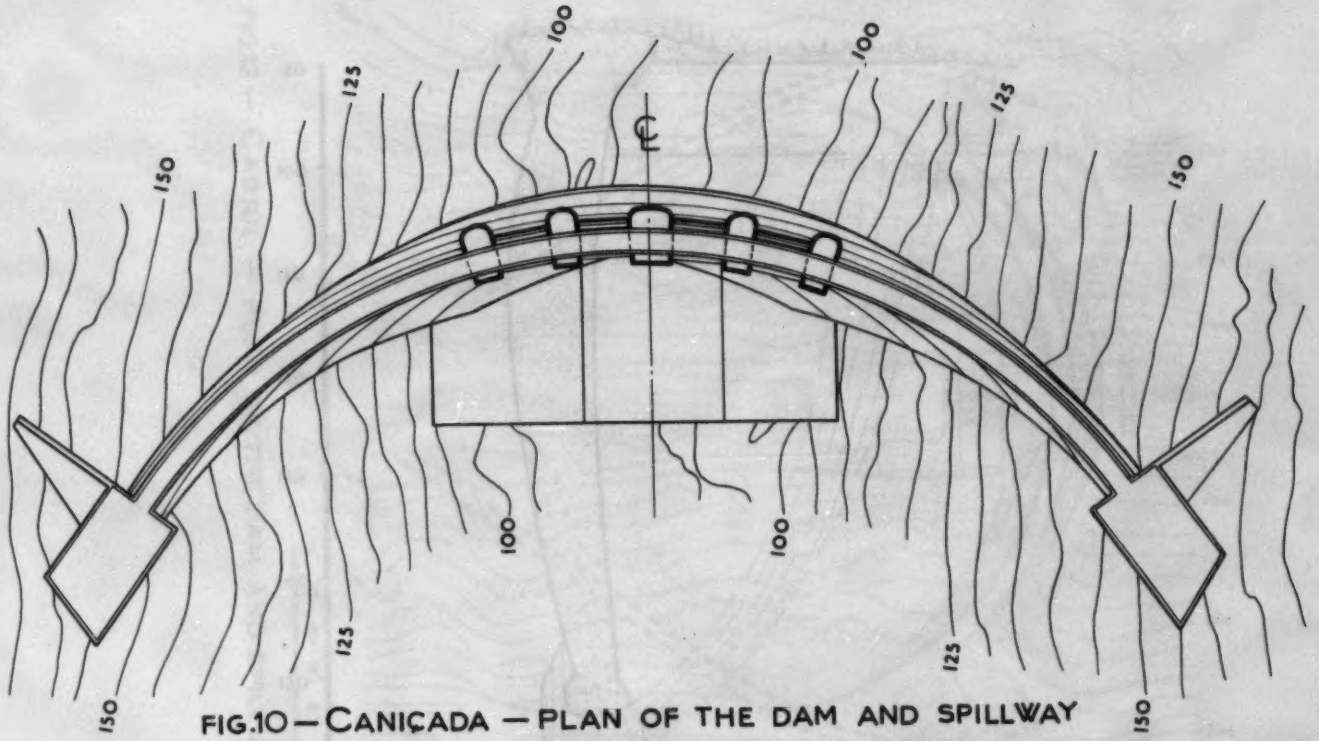

FIG.10 — CANIÇADA — PLAN OF THE DAM AND SPILLWAY

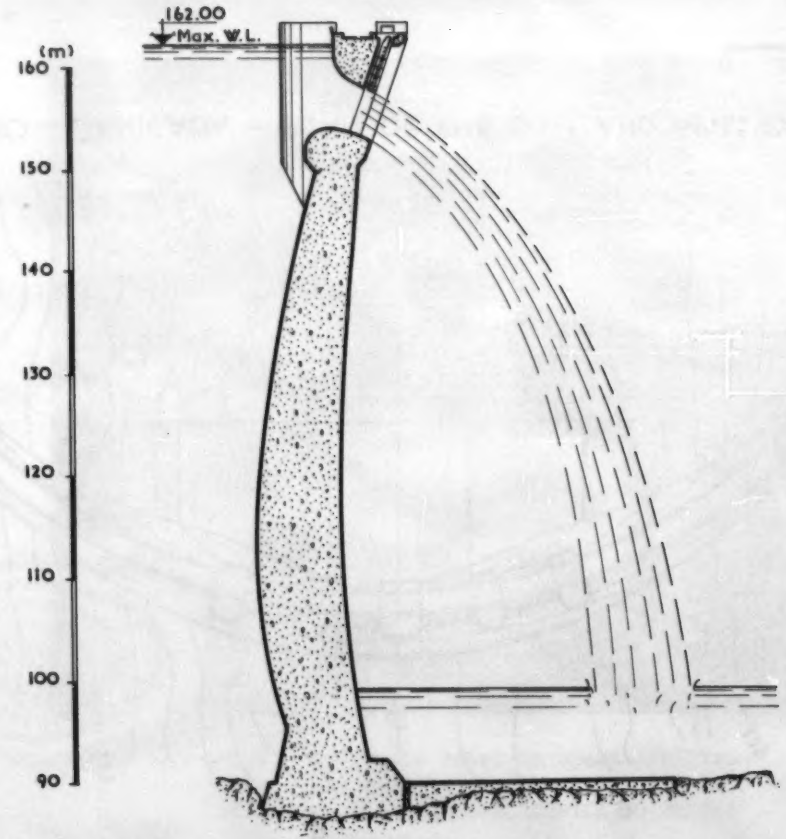

FIG. 11 — CANIÇADA — CENTRAL PROFILE OF THE DAM AND SPILLWAY

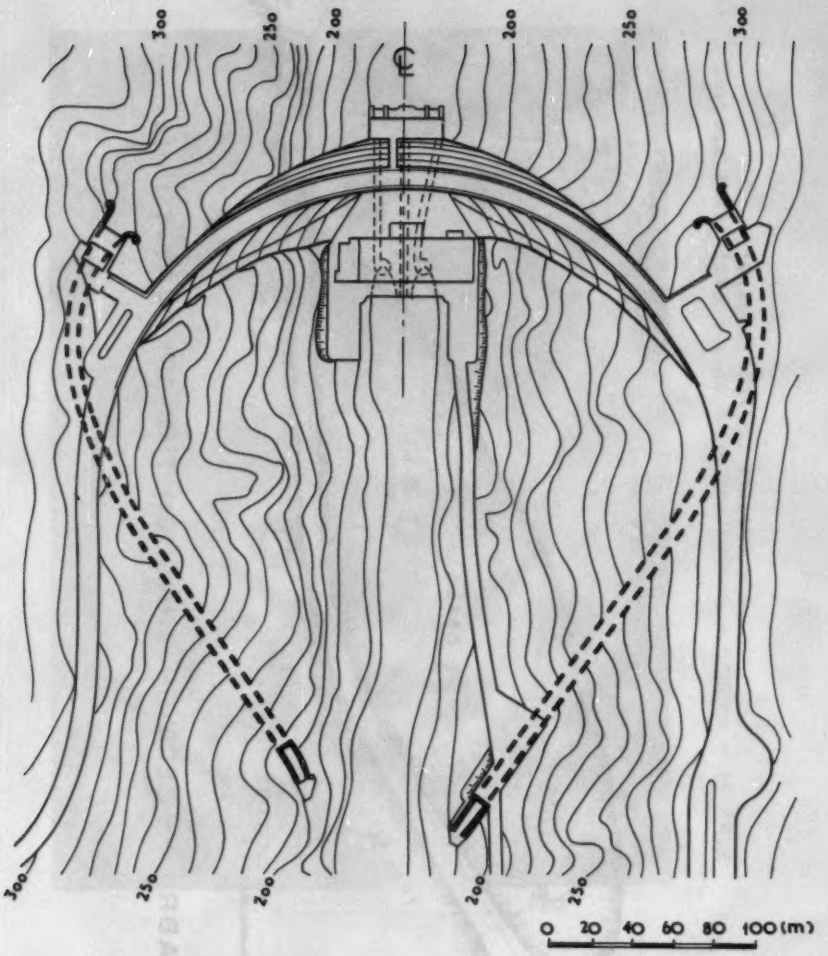

FIG.12 — CABRIL — PLAN OF THE DAM AND SPILLWAY

FIG.13 — CABRIL — SECTION THROUGH THE SPILLWAY TUNNEL

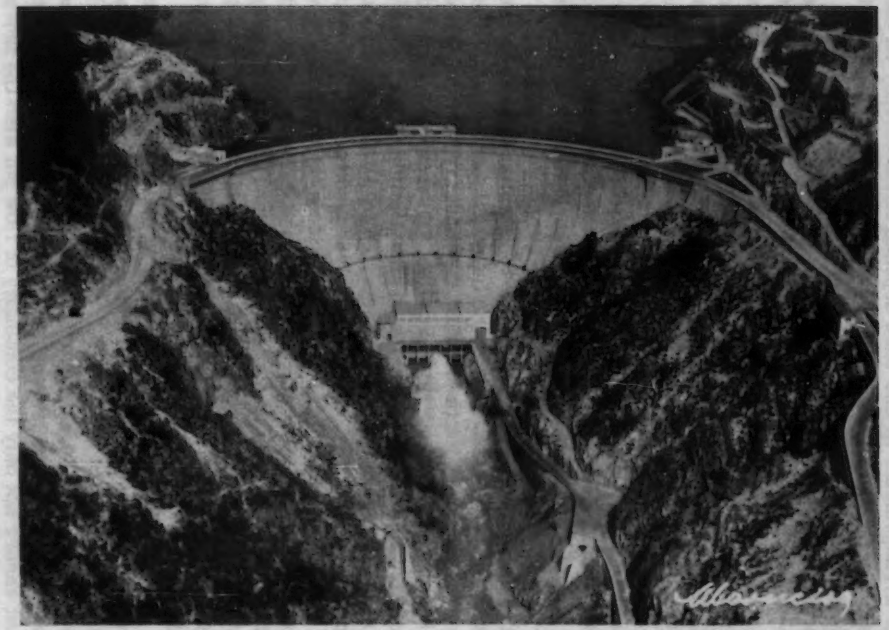

Fig. 14 - CABRIL - Downstream view of the dam, power station and outlets of the tunnels.

to the conclusions drawn at the 5th Congress on Large Dams when it was stated that the choice of the type of dam to build must always be considered case by case and depends on many factors besides the specific one of the actual design of the dam. In effect, the solution to be adopted for the dam and flood discharge arrangements in this case depends fundamentally on the location of the power station and on the problem of the restitution of the water from the turbines downstream.

Without discussing the various possible solutions which were considered before arriving at the final one it may be said that studies were conducted envisaging an underground power station with the restitution of the turbine water upstream of the flood discharge zone. This solution has the advantage of minimising the loss of head during floods and of making this restitution in a zone where there is no disturbance. It was possible to conjugate these requirements in a project involving the building of a thin arch dam where the floods are discharged over the whole crest through four openings each 20.00 x 15.00 m (65.62 x 49.21 ft). The throwing of the jet downstream is satisfactorily made by means of a ski-jump spillway* under which the station tunnel discharges (Figs. 15, 16).

As a result of this layout it was necessary to establish two independent structures having different elastic characteristics: the arch dam of the best possible design within the limits imposed by the topography of the site and with a crest shape in plan and profile determined by means of hydraulic model tests and conveniently arranged for the installation of the radial gates; and the ski-jump spillway which is to be supported by a buttress structure.

The ski-jump spillway design was carefully checked through elaborate hydraulic model tests (Figs. 17, 18). During these tests the shape of the spillway was thoroughly studied, specially the convergence in plan of the side walls, the inclination of the slab, the angle of throw and the special shape to be given to the dentated sill so as to have the best possible aeration of the jet and, consequently, the best energy dissipation. The jet is thrown centrally in the valley and is widely dispersed as a result of the action of the dentated sill. Velocity distribution in the river is satisfactory, the greater part of the flow being concentrated in the centre with lower velocities near the banks as required.

As regards constructional details particular reference should be made to the longitudinal expansion joint between the crest of the dam and the buttress slab which must ensure not only the continuity of flow but also the independent movements of the two structures. The crest of the dam supports the piers whose shape have been established to permit perfect hydraulic flow; these piers are reinforced concrete structures to which the control gates of the radial type are tied. Results of the model tests now in course will give an indication of the stresses in every part of this delicate zone of the arch structure to enable the best constructional solutions to be chosen.

Before finishing attention is drawn to the fact that although contribution of

* After the above project had been designed the writer has learnt through the kindness of eng. Carlo Semenza of SADE (Italy) that an analogous solution for the flood discharge was studied, but not built, with regard to the Sottosella arch dam.

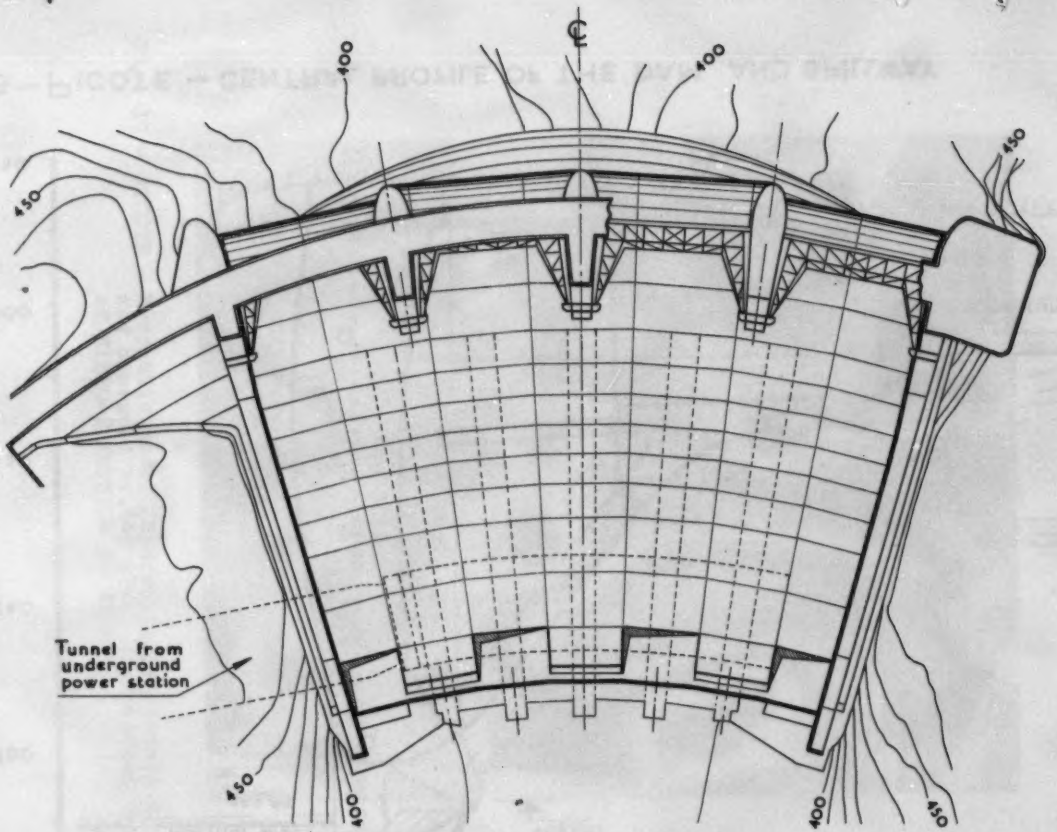

FIG.15 – PICOTE – PLAN OF THE DAM AND SPILLWAY

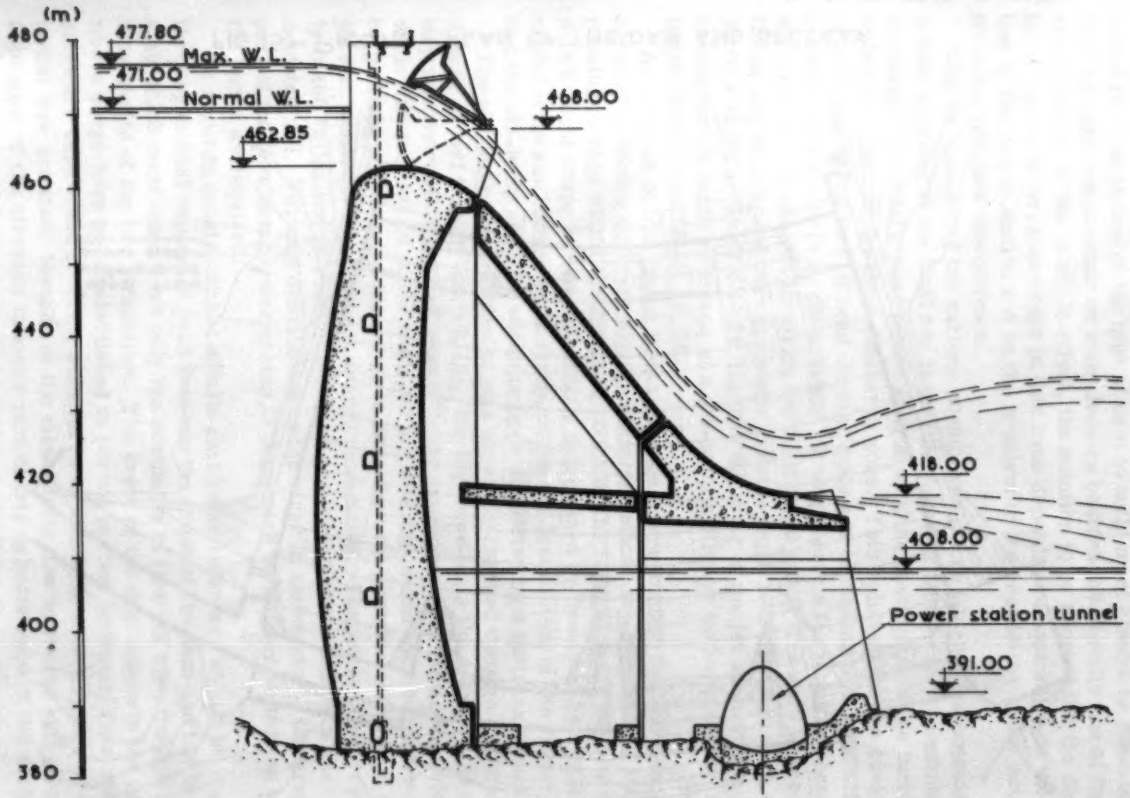

FIG.16— PICOTE – CENTRAL PROFILE OF THE DAM AND SPILLWAY

Fig. 17 - PICOTE - Hydraulic model (Hydraulic Laboratory directed and operated by the Civil Engineering Department of HIDRO-ELÉCTRICA DO ZEZERE).

Fig. 18 - PICOTE - Hydraulic model in operation (Hydraulic Laboratory directed and operated by the Civil Engineering Department of HIDRO-ELECTRICA DO ZEZERE).

Portuguese engineers to the development of the technique of arch dams is very small when compared with that from other parts in the world, it should be stressed however that the studies which have lately been carried out in Portugal on overflow arch dams have been conducted within the best technical principles of economy and safety.

NOTE:—The works of Cabril, Bouçã and Castelo do Bode constitute the hydroelectric development in the river Zezere of which the Hidro-Eléctrica do Zezere are concessionaires; those of Venda Nova, Salamonde and Caniçada in the rivers Cávado and Rabagão form part of the concession held by the Hidro Eléctrica do Cávado; the dam which is being built at Picote is included in the development of the Douro river scheme awarded to Hidro-Eléctrica do Douro. Castelo do Bode, Venda Nova and Salamonde dams were designed by the French consulting engineer A. Coyne; Caniçada dam was designed by the Civil Engineering Department of the Hidro-Eléctrica do Cávado; Cabril and Bouçã dams as well as Picote were designed by the Civil Engineering Department of the Hidro-Eléctrica do Zezere.

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Paper 991 PO 3

Journal of the
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ARCH DAMS: THEORY; METHODS, AND DETAILS OF JOINT GROUTING

A. Warren Simonds,[1] M. ASCE

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno De Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested readersshould watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

In the construction of arch dams, the present practice of the Bureau of Reclamation is to build the structure in blocks which are bounded by radial joints keyed to resist shear deformations. The purpose of the joints is to permit shrinkage of the concrete, due to dissipation of the setting heat of the cement, to take place without developing irregular cracks throughout the

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1. Structural Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

structure. In order for arch action to take place, it is essential that no structural discontinuities exist at the time the reservoir load is applied. To make possible the development of arch action during the early stages of reservoir filling and to improve the stress distribution for the "in-service" condition of the dam, a procedure has been developed whereby the contraction joints of arch dams are pressure-grouted, thereby making the structure monolithic. Metal seals capable of expansion and contraction are installed around the peripheries of the joints or areas to be grouted so as to hold the grout in the joint. Special systems of piping with outlets to the faces of the joint are used for conveying the fluid grout into the joints. A technique has been developed whereby abnormal stresses due to grout pressure can be controlled so as not to damage the structure, and an effective job of grouting will result.

INTRODUCTION

An arch dam, as referred to in this paper, is a solid concrete or masonry dam which is curved upstream in plan, and, because of its shape, carries the reservoir waterload partly by its own weight and also by transmitting the remainder of the waterload to the canyon walls by arch action. In order for arch action to develop in the structure, it is essential that no structural discontinuities exist at the time the waterload is applied as the reservoir fills. Sound abutments which are susceptible to only minor deformation are necessary requirements for an arch dam.

An intensive study of the structural action of arch dams was inaugurated in the United States during the early part of this century. It had been realized by designers that contraction joints were valuable for the control of cracking which oftentimes resulted in undesirable leakage. The opening and closing of contraction joints had been observed at the Salmon Creek Dam in Alaska, and an extensive investigation was made of the behavior of that structure due to changes in temperature and waterload conditions.

As the result of these observations, it was concluded that arch action did not develop until after sufficient load had been applied to the structure to close any open contraction joints and the blocks forming the voussoirs of the arch were in bearing. It was concluded that the two main reasons for the phenomenon that an arch dam does not act as an arch before considerable load has been applied to it were: (1) the shortening of the arch rib due to shrinkage of the mass concrete; and (2) the bond of the concrete of the structure to the foundation.

It is obvious that if an arch dam could be constructed in such a way that arch action would begin with the application of reservoir pressure on the upstream face of the dam, a better distribution of stress throughout the structure would result. To obtain this condition a suitable means of bringing the faces of the blocks of the arch into contact before the application of waterload was sought. Two methods were developed: (1) by leaving open slots between the blocks of the arch dam and constructing closing plugs at a time of no reservoir load on the arch and when minimum temperature conditions of the concrete prevailed; and (2) by grouting the open joints under pressure at a time of maximum joint opening before the reservoir waterload was applied.

Typical early examples of these approaches to closures of the arch elements were at Arrowrock Dam where shafts 6 by 6 feet were constructed across the joints and later filled with lean concrete; at Gerber, Waterville,

and Ariel Dams where closure plugs were used; and at Bullards Bar, Cushman No. 1, Pocoima, and Lake Spaulding Dams where the contraction joints were grouted. It was during the construction of these latter dams when the use of embedded piping systems installed especially for grouting contraction joints was developed.

History of Joint Grouting Development by Bureau of Reclamation

The first dam in which the contraction joints were grouted to be built by the Bureau of Reclamation was Gibson Dam on the Sun River Project, Montana, which is shown in Figure 1. This dam is a concrete arch having a structural height of 195.5 feet, base width at the crown section 87 feet, crest width 15 feet, crest length 960 feet. Its volume in cubic yards is 167,500. The dam was built during the period of 1926-8, and the contraction joints were grouted in 1930. Because this dam was the forerunner of a number of large dams to be built by the Bureau, an extensive program of research on structural behavior of the arch was undertaken. This included studies of temperature, deflection, and hydrostatic uplift pressure on the base of the structure. Special installations were made at the dam to check various design theories and assumptions. Studies were made of the joint treatment, and elaborate installations for grouting the contraction joints were made.

The grouting of the contraction joints presented a number of problems, and a systematic study of these was inaugurated by the Bureau in 1927. It was realized that when a contraction joint having considerable area was filled with fluid grout, an enormous total pressure could be built up on the faces of the joint by applying even small increments of pressure on the fluid grout. If not carefully controlled, the pressure on the grout could be increased easily until there was danger of damaging the structure. Since an arch dam built with radial contraction joints is designed to be safe when waterload is applied at its upstream face, such a structure may not be stable when excessive pressure is built up in a contraction joint. Excessive pressure in a joint may force the joint open by bending the blocks laterally against neighboring blocks; it may shear a block off along the plane of a horizontal construction joint, or it may force the adjacent blocks to bend upstream, thereby causing tensile cantilever stresses at the downstream face with resultant cracking.

The problems considered in the study of contraction joint grouting procedure included the following:

Opening of contraction joints
Unit weight of grout
Air entrapped between grout outlets
Tangential deflection of crown cantilever
Effect of grouting different joints
Radial deflection and tension areas
Concrete temperature measurements
Best time to grout joints

The following is a summary of the results of this investigation. It was concluded that joints opened between 1/16- and 1/8-inch could be grouted readily. The unit weight of the fluid grout was dependent on the water-cement ratio used. Under normal conditions, the fluid grout used weighed about 105 pounds per cubic foot. In the systems of piping and outlets first used, there was a possibility of entrapping air between the grout outlets; however, by injecting

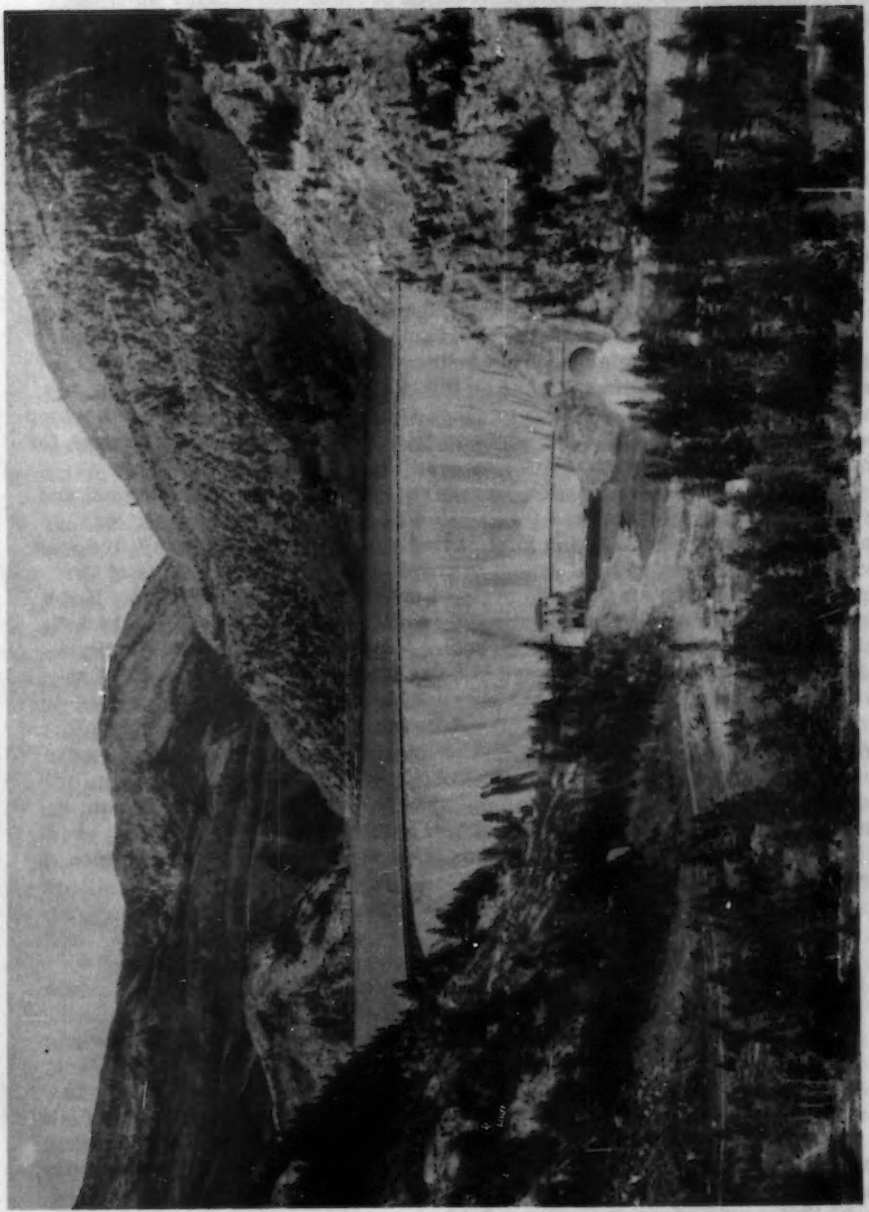


Fig. 1 General View of Gibson Dam, Sun River Project, Montana

the grout slowly, the fluid grout could settle in joints and the air could escape through the outlets in the upper part of the systems. From the study of the deflection of the central blocks in tangential directions when the joint at the cantilever section was filled with fluid grout, it was concluded that the blocks would deflect sufficiently to close the adjacent ungrouted joints. This difficulty could be controlled by filling the adjacent joints with water so as to balance the grout pressure. Studies were made also of grouting the joints along the abutments where the heights of the joints near the ends of the dam became progressively less. In order to control the upstream deflection of the blocks which would be accompanied by the development of tensile cantilever stresses at the downstream face, a system of observations using theodolite reference lines, was evolved. Studies of temperature conditions were made and charts showing the range of temperature variation were plotted so that the time of minimum temperature could be determined. Since there was no provision for artificial cooling of concrete at Gibson Dam, the temperature of the mass concrete was dependent on weather conditions, and the exact minimum could not be determined except by observation. However, it appeared that during the late spring, the temperature would be close enough to the minimum to permit a satisfactory condition for grouting.

Radial Joints

The first designs of contraction joints for arch dams provided for radial joints to be constructed at some specified interval. In constant-radius arch dams, the joints were laid out radial to the axis. In variable-radius and constant-angle types of arches, the joints were designed either curved in plan or with two straight sections at an angle, connected by a curved section so that the upstream part of the joint was radial to either the axis or the upstream face, and the downstream part was radial to the downstream face. In either case, it has proved desirable to construct the joints with keys.

The joint keys developed by the Bureau of Reclamation have now been standardized and are used in the construction of all solid concrete dams. Figure 2 shows the arrangement of keys at the face of one of the blocks of Shasta Dam. Although Shasta Dam is a curved gravity dam, the keys as developed there, are the standard design of the Bureau for arch dams. The keys are dimensioned so as to give equal shearing resistance between the two adjacent joints. This standard key offers the least obstruction to the flow of grout, provides a greater theoretical shear value, eliminates sharp corners which are cracked frequently by form removal, and improves the reentrant angles conducive to crack development resulting from volume change.

As now developed, the fluid grout is injected into the contraction joints through an embedded system of piping and circular outlet units which are spaced uniformly over the face of the joint. The outlet units comprised of two modified electrical conduit boxes, are connected by tees to vertical riser pipes which are attached to a supply header. This system is installed in the high block. The supply header is mounted on metal supports and the outlet units without covers are attached to the concrete forms. After embedment when the forms are stripped, the covers of the units are attached to their companion members before the concrete of the corresponding lift in the adjacent low block is placed.

In the development of grouting systems, a study and some experimenting have been done to determine a satisfactory height for a grouting lift.

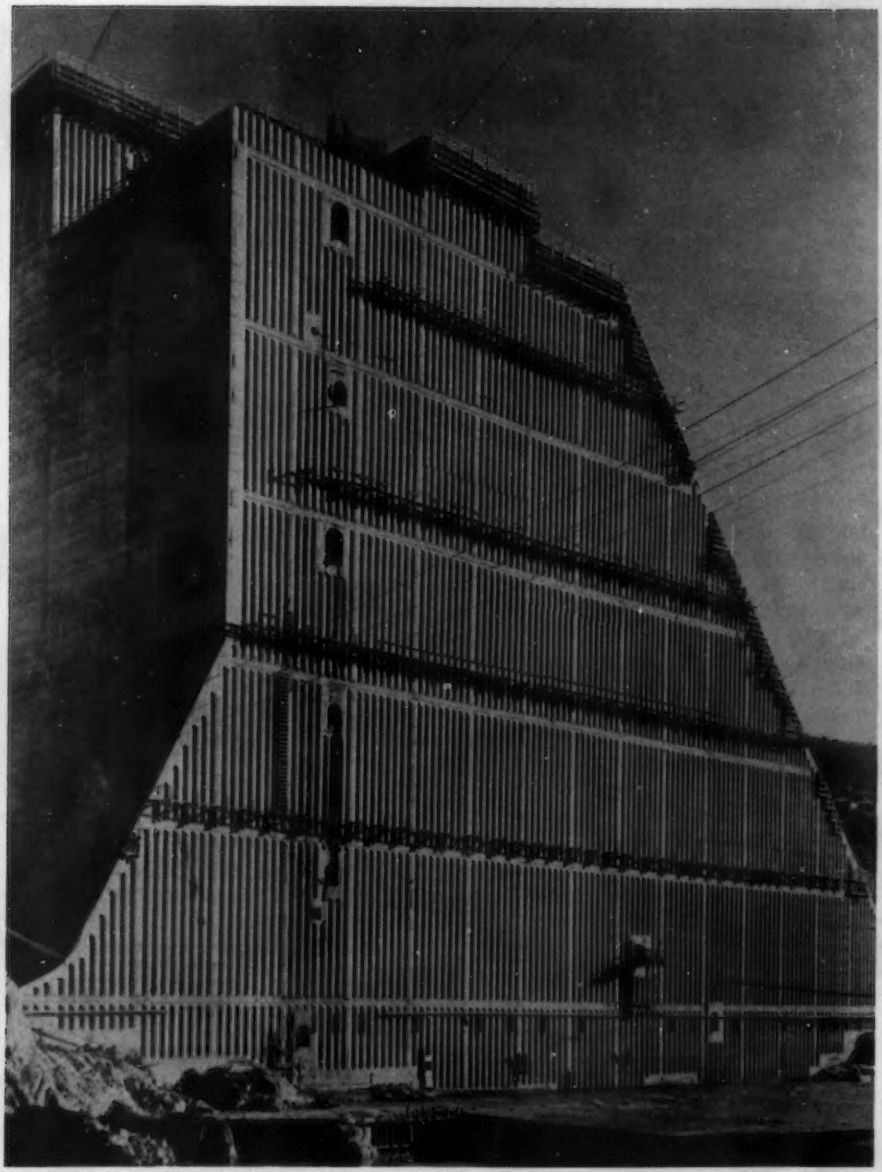


Fig. 2 Face of Joint in Shasta Dam Showing Keys, Central Valley Project, California

Originally the grouting was done from the bedrock to the top of the dam through heights ranging from 25 to 175 feet. Because of the difficulties encountered in grouting through the greater heights, the standard practice now is to provide for grouting through vertical lifts of about 50 feet. A moderate variation from this height does not affect the grouting procedure.

In order to contain the grout in the joint, metal sealing strips around the periphery of the joint are installed. Horizontal sealing strips are used to divide the joints into vertical lifts. The sealing strips are of lightweight metal, preferably either of soft tempered copper, 24-ounce weight per square foot, 0.032-inch thick, or annealed stainless steel No. 20 gage, 0.0375-inch thick. The seals are usually installed in an M- or Z-shape. Connections are made by brazing or welding.

A more recent development consists of installing a vent groove at the top of each grouting lift just below the horizontal seal. This provides for the escape of air and excess water from the grout during the filling of the joint. When difficulties are encountered and it can be anticipated that regrouting at the joint will be necessary, the vent can be used for injecting grout into the joint.

Theory of Grouting Contraction Joints

When arch dams are to be constructed with radial contraction joints which are to be grouted, a stress and stability analysis should be made of the structure before grouting operations are initiated. The blocks of an arch dam are designed to be statically stable under the waterload after the contraction joints are grouted. Before the joints are grouted, severe stress conditions may be set up due to the pressure of fluid grout in the joints.

The forces acting on the radial faces of a block in an arch dam due to fluid grout in a contraction joint vary with the procedure with which the grouting is done. If an individual joint is to be grouted alone, the pressure of the fluid grout in a circumferential direction may be sufficient to force the joint open by bending the blocks over against their adjacent neighbors, thereby squeezing the adjacent joints shut. If a series of joints is to be grouted simultaneously, the excessive bending of the blocks in a circumferential direction may be controlled and the bending stresses held to a minimum. In small arch dams, the usual practice is to grout all joints in one lift from abutment to abutment in one operation. In larger arch dams the joints are grouted in groups; the abutment groups are grouted first and the center groups last.

Where the planned procedure for grouting the contraction joints is to grout all the radial joints from abutment to abutment, different stress conditions are developed. Consider the forces acting on an individual block of arch dam due to grout pressure in the radial contraction joints as shown in Figure 3. The resultant of the forces acting on the faces of an individual block acts in an upstream direction and tends to bend the block as a cantilever upstream. The resultant may be sufficient to cause high tensile cantilever stresses to develop on the downstream side of the structure. These can be controlled to a large extent by a partial filling of the reservoir to balance the resultant of the grout pressure.

Since the pressure of fluid grout in the contraction joints of an arch dam may cause considerable damage if not adequately controlled, an important factor that must be considered in the design of a grouting system for an arch dam is the maximum expected grout pressure. Laboratory investigations of

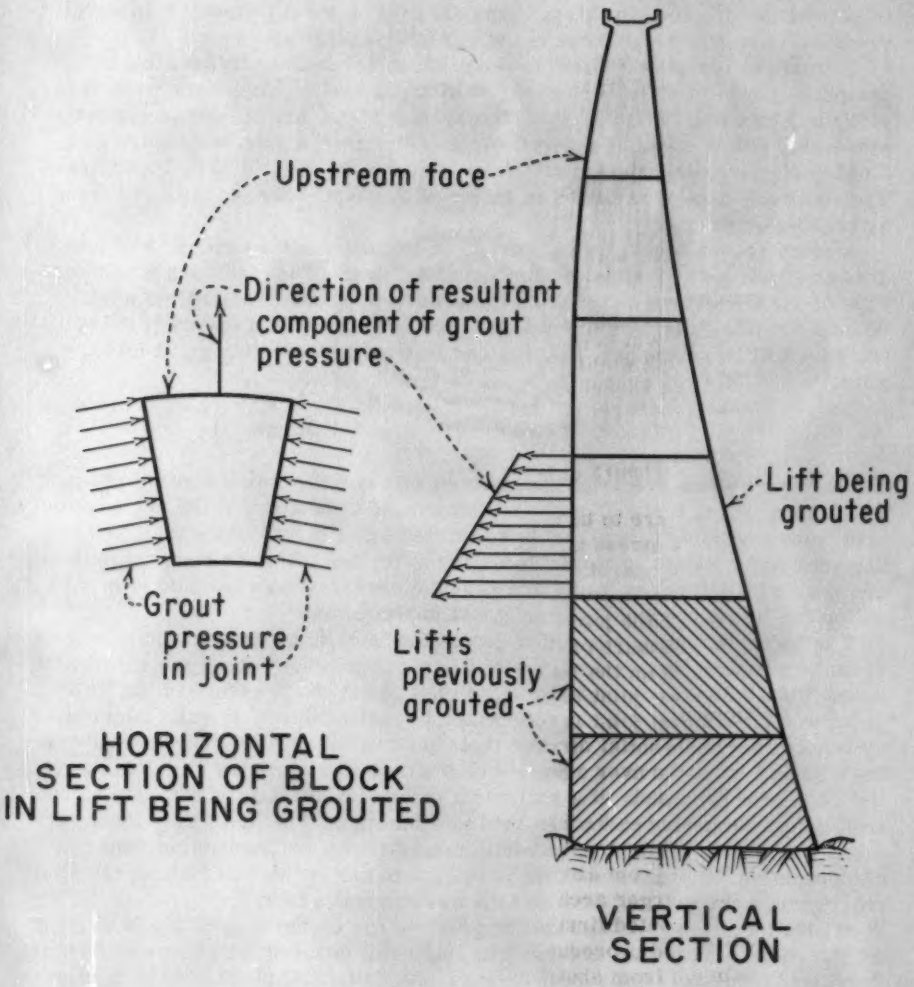

Upstream face
Direction of resultant component of grout pressure
Lift being grouted
Grout pressure in joint
Lifts previously grouted
HORIZONTAL SECTION OF BLOCK IN LIFT BEING GROUTED
VERTICAL SECTION

Fig. 3 Forces Acting on a Block of an Arch Dam Due to Grout Pressure

neat cement grouts have indicated that when they are allowed to set under pressures of 30 to 50 psi good films of grout will result. Most contraction joint grouting systems are therefore presently designed so that excessive stresses will not occur when a grouting lift filled with fluid grout has pressures of 30 to 50 psi at the top of the lift. This pressure is usually measured at the vent at the top of the lift being grouted.

The effect of pressures in the contraction joints is to cause the blocks of the arch dam to deflect and, because of this, observations for deflection may be used beneficially for controlling grouting operations. There are two general types of observations: (1) upstream deflection of the dam, and (2) change in width of joint opening. The first type of observation consists in measuring the deflection of some point on the top of the dam at the crown cantilever section with reference to a fixed line normal to the radius of the axis at that point. A theodolite is used in making observations of this type.

The second type of observation consists in measuring the change in width of joint opening. This is done by means of dial gages which register to 0.0001 inch. The gages are mounted usually across the contraction joints at the top of the lift being grouted. An increase in the width of joint opening at the top of a 50-foot-high lift being grouted should be not greater than 0.0200 inch under normal conditions. Occasionally when the grout pressure gets out of balance between joints, one joint will open while an adjacent joint will close. Dial gages will indicate this occurrence and the pressures on the grout can be corrected.

Development of Block Construction

In order to control temperature stresses so as to prevent cracking, it has been found that contraction joints or "formed cracks" must be provided at frequent intervals if cracking of the concrete is to be avoided. Originally, the joints were spaced at intervals of 30 to 60 feet. When dams are of exceedingly large sections such as Hoover, Shasta, and Hungry Horse Dams, it is necessary to provide joints in two directions to control cracking.

In the design of Hoover Dam, limitations on maximum and minimum block sizes influenced the contraction joint design. If the blocks were too large, cracking might occur within the blocks. On the other hand, if the blocks were too small, shrinkage might be so slight that the contraction joints between blocks would not open far enough to take grout. After a study of the conditions under which the grouting would be done, a two-way system of joints extending in radial and circumferential directions was developed. These joints were spaced so that they would be not less than 25 feet apart at the downstream face of the dam and 65 feet apart at the upstream face.

From stress considerations, it was desirable to have continuous radial joints and staggered circumferential joints as shown in Figure 4. All blocks extended vertically from the rock contact at the base of the dam to the intersection with the upstream or downstream face or the top of the dam. The lower part of the large radial joints in the central area of the dam were divided into two sections for convenience in grouting. The circumferential joints were designed so that there were three groups of joints between the abutments in each grouting lift, except at the extreme bottom and top of the dam where there were one and two groups respectively.

In the more recent designs of grouting systems, a number of improvements have been incorporated in the plans based on difficulties which had been

Fig. 4 Block Construction at Hungry Horse Dam, Hungry Horse Project, Montana

experienced previously. These include a more convenient arrangement of the galleries where used, a more effective system of vents at the top of the lift being grouted, and the installation of header drains to be used in cleaning the joints and systems prior to grouting.

A typical plan of a grouting system for a transverse joint is shown in Figure 5. Inasmuch as it is not practicable to provide special galleries in each 50-foot grouting lift in most arch dams for grouting the contraction joints only, the grouting is generally done from temporary cat walks installed on the downstream face. When a suitable gallery is constructed near the foundation of an arch dam for the foundation grouting and drainage, the headers of the piping systems for the contraction joints may be arranged so that the grouting of the lower lifts of the joints can be done from this gallery. In either case, a temporary supply line is constructed from the grout pump to connect to the supply headers of the contraction joints. The grout is injected through the supply headers and the embedded piping systems into the joints. As the grout fills the joints, any water in them is forced ahead of the grout into the vent at the top of the lift. At the downstream end of the vent, a length of pipe about 5 feet long is installed vertically to serve as a temporary standpipe. A pressure gage and valve are installed at the upper end of the standpipe.

The grouting systems for the longitudinal joints are in sections measuring 50 by 50 feet because of the discontinuity that results from the staggered joints in the block pattern. A typical elevation of the grouting system across one block is shown in Figure 6. The grout is injected into the longitudinal system through the supply header and enters each section of the joint through the piping system. Outlets are spaced at intervals along the horizontal keys. The excess water and thin grout are drained from the vent groove at the top of the lift.

If some essential header should become plugged accidentally during construction, and the header could not be utilized, the system is designed so that an alternate header could be used. The plan of the longitudinal headers is shown in Figure 7. Normally the grout would be injected into the supply header; if the supply header could not be used, the joints could be grouted through the 1-1/2-inch return header.

In grouting the longitudinal joints, the critical areas where the pressure must be accurately controlled, are the partial lift joints terminating the downstream face and the joints nearest the upstream face of the dam. In order to provide adequate control for grouting these joints, separate grouting systems should be installed for each set of joints. The grouting of these joints can be done at the same time the regular longitudinal joints in the zone are grouted but individual control of each partial lift is necessary.

The vent system of the longitudinal joints is shown in Figure 8. This system is arranged so that each group 50-by-50-foot sections has a vent return. Riser pipes serve as standpipes to hold hydrostatic pressure on the grout in the joint, similar to the function of the risers on the vents in the transverse system.

Cement Used in Contraction Joint Grouting

The cement used in contraction joint grouting includes normal, modified, and low-heat portland cements, and occasionally special cements such as high-early-strength cement and special grinds of oil-well-grouting cement. The choice of type of cement used depends on field conditions in which

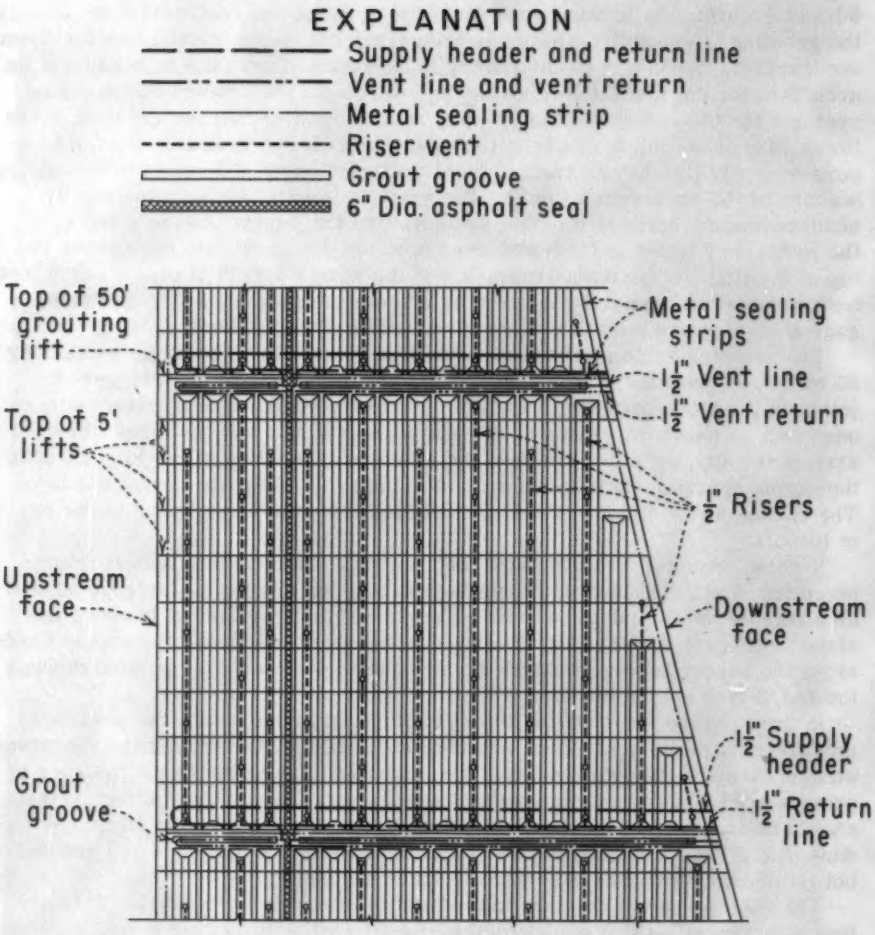
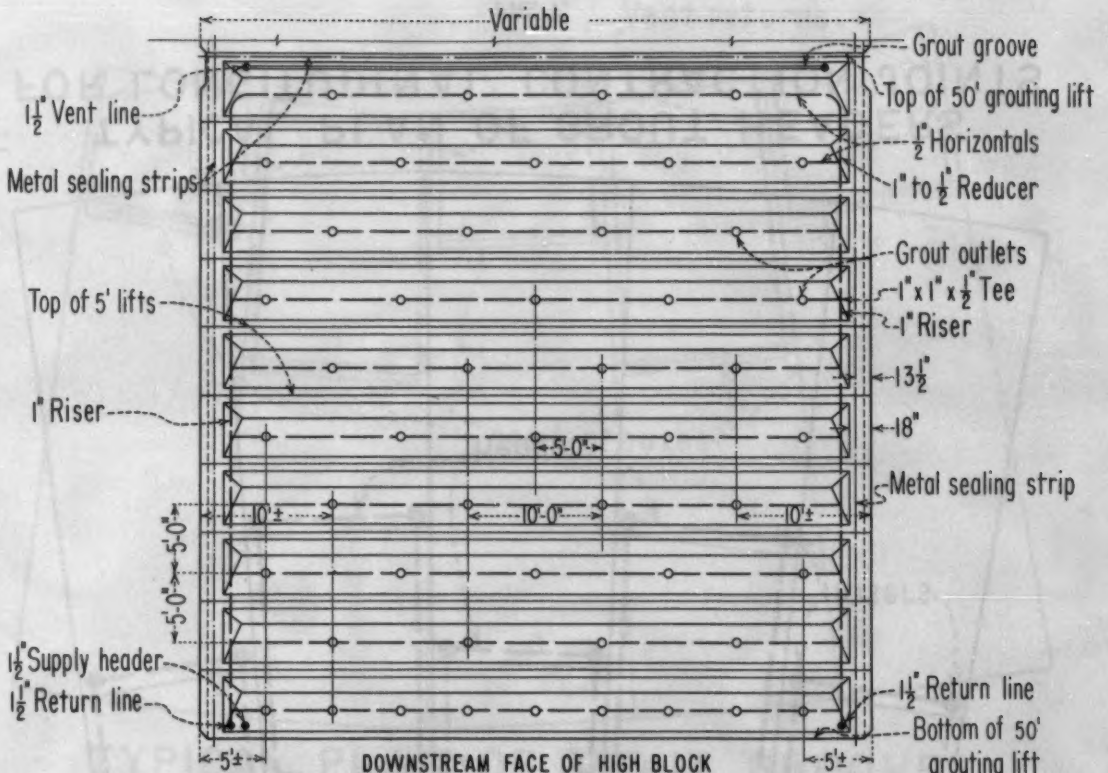


FIG. 5.

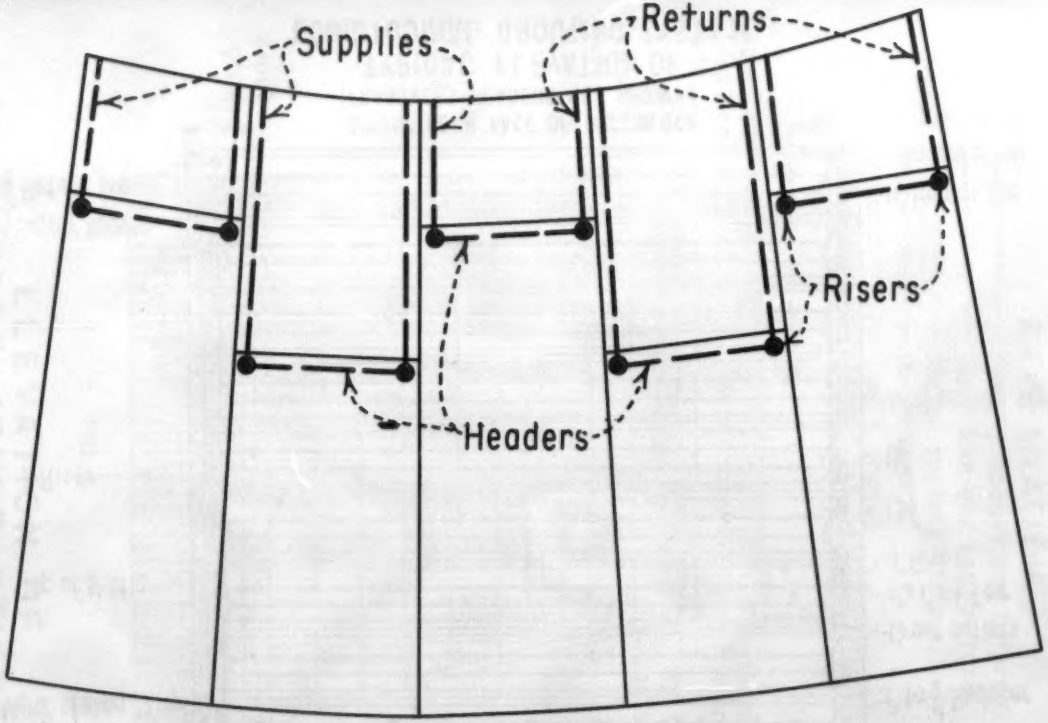
ELEVATION OF TRANSVERSE CONTRACTION JOINT

SHOWING HEADER AND RISER PIPE LAYOUT



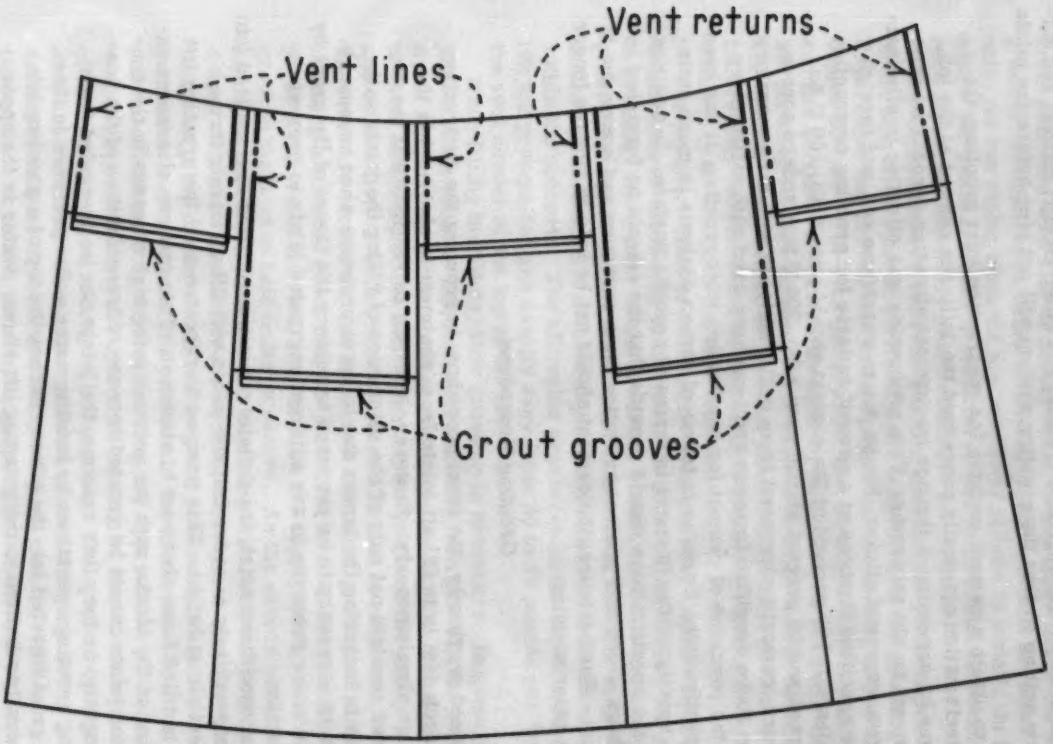
TYPICAL ELEVATION OF LONGITUDINAL GROUTING SYSTEM

FIG. 6.



TYPICAL PLAN OF GROUT HEADERS FOR LONGITUDINAL CONTRACTION JOINTS

FIG. 7.



TYPICAL PLAN OF GROUT GROOVES AND VENT LINES FOR LONGITUDINAL CONTRACTION JOINTS

FIG. 8.

transportation, time of set, and ability to seal leaks in inaccessible locations are involved. In general, of these types, modified and low-heat portland cements and oil-well grouting cements have been the most satisfactory. The desirable qualities of a cement suitable for contraction joint grouting are: sufficient fineness for grouting joints where the width of opening is small; freedom from tramp iron, unground clinker, and other foreign matter; and sufficiently slow setting so that the grouting system will not plug before the joints are filled.

In order to obtain a cement suitable for contraction joint grouting, the Bureau of Reclamation originally processed the mill-run cement at the job. This was done by screening it through mechanical vibrating screens or processing it through an air separator. This processing was effective in eliminating tramp iron, unground clinker, lumps due to warehouse set, and foreign matter. The specified fineness of a cement suitable for grouting contraction joints provides that 100 percent of the cement should pass a No. 100 U.S. standard screen and 98 percent should pass a No. 200 U.S. standard screen. In recent years, cements purchased from mills equipped with air separators have met the above specified fineness and have been used with satisfactory results. In the purchase of cement for contraction joint grouting, it has been found that specifying the fineness in terms of screen analysis is more satisfactory than specifying the fineness in terms of specific surface. In addition to fineness, the specifications should provide that the cement be furnished in waterproof bags which will prevent hydration from exposure and lumps due to warehouse set. Furthermore, the cement should not be held in storage longer than 90 days prior to using.

Grouting Procedure

As mentioned previously, the ideal procedure in grouting the contraction joints of an arch dam is to fill all joints from abutment to abutment to the same elevation simultaneously. However, it should be realized that the procedure adopted consists not only of the mechanics of filling the joints with grout but also in balancing the forces due to grout pressure when necessary. Therefore, it is necessary to be prepared to balance the thrust of the grout by holding water under pressure in any adjacent ungrouted joints to prevent excessive deflection.

When ideal conditions exist, the definite quantities of grout are injected into each joint consecutively in order that the joints will fill uniformly through small differentials in height. This process is repeated until the grouting lift of all joints is filled from abutment to abutment. This process eliminates excessive tipping of the blocks with the accompanying high stresses in the concrete. If some joints cannot be grouted because of irregularities of the construction program, or for other reasons, the joints can be prevented from closing during grouting operations by holding water under pressure in them.

When the grout injected into the joints reaches the top of a grouting lift, water is generally admitted to the grouting lift above. Water in the upper grouting lift serves several purposes: first, the water enables the grouting engineer to check for leaks past the horizontal sealing strip into the lift above by flushing water from the supply and return headers; second, the water reduces the unbalanced pressure on the sealing strip caused by the grout below the sealing strip, thereby retarding the development of possible leak; third,

the water tends to spring the joint open permitting a more effective spread of the fluid grout in the lower lift.

In normal grouting operations, two sets of temporary piping are necessary: one set to supply grout to the joints, and the second set to supply water to the adjacent joints, when a water curtain is needed to balance the thrust of the grout pressure, and also to supply water to the upper lift of the joints being grouted. In small arch dams where the joints are built in radial directions only, the two piping systems are relatively simple to install. In large dams where the joints are built in transverse and longitudinal directions, the necessary piping systems may reach extensive proportions.

The equipment required for grouting contraction joints depends on the size of the dam and on the accessibility of the headers of the grouting systems. Experience has proven that an equipment unit consisting of one air-driven duplex 10- x 3- x 10-inch grout pump, one 20-cubic-foot-capacity grout mixer and one 20-cubic-foot-capacity agitator sump can meet easily the requirements for grouting 100,000 square feet of joint area in an 8-hour shift, if conditions are normal. In grouting contraction joints, costly delays caused by equipment failure may be minimized by providing a complete standby unit of equipment, which may be placed in service when needed.

The pressure used in contraction joint grouting depends upon the physical dimensions of the blocks of the dam. In general, the higher the pressure used in grouting, the denser is the resulting grout film. The maximum allowable grout pressure is the pressure that the blocks of the dam will withstand before deflecting excessively, or creating undesirable tensile or shearing stresses within the concrete. The allowable pressures at the top of a 50-foot grouting lift for present designs usually vary from 25 to 60 pounds per square inch at the vent header at the top of the lift.

When filling the joints, slow grouting is necessary. In grouting a series of joints, if the grout is injected intermittently, the grout has a better opportunity to settle in the joints, and the excess water and thin grout can be bled from the vent at the top of the lift. When all the joints of the group have been filled and when a flow of thick grout has been obtained at each vent, the valves on the vents are closed and the maximum allowable grout pressure is applied by injecting additional grout into the joint. As soon as the maximum allowable grout pressure is reached, the valve on the supply header is closed and the grout in the joint is held under pressure. As the excess water is slowly squeezed out of the grout into the adjoining concrete, the pressure at the vent drops. When the pressure drop occurs, more grout must be injected into the joints until the maximum allowable grout pressure is again reached. The process is repeated until the joints hold the maximum allowable grout pressure about 30 minutes without showing any appreciable pressure loss.

Grouting operations are governed by careful observation of the deflection of the blocks while grouting is in progress. Dial gages registering to 0.0001 inch are valuable. These are mounted across the contraction joints, to observe the amount of joint spreading. The magnitude of the permissible deflection of the blocks in a dam depends upon the dimensions of the blocks. In general, a contraction joint under pressure should not increase in width more than 0.02 inch at the top of a 50-foot lift. Undesirable tensile or shearing stresses may develop if greater spreading occurs.

Results

At the time Shasta Dam was being designed, the question of relative efficiency of plane versus keyed grouted contraction joints in mass concrete came up, and a laboratory program of testing grouted joints was undertaken.

The specimens consisted of 10- by 25-inch cores drilled from concrete blocks 25 inches square and 48 inches high. These blocks were prepared by bolting two triangular blocks 48 inches high together with a 0.05-inch separation shim between them and grouting this 0.05-inch space. This method produced a grouted joint at an angle of 45° with the axis of the cores. Three cores were drilled from each of three blocks after being grouted at 50, 30, and 10 psi with pressures maintained for 30 minutes. One core from each block was drilled and tested when the grout was 28 days old, and the two remaining cores were drilled and tested when the grout was 90 days and 123 days old. Six cores were also drilled from a control block cast without a joint and tested at the same ages as the grouted cores.

The results of these tests indicated that if the joint between the two blocks of concrete was completely filled, the strength will approach the strength of concrete. While most of the specimens failed along the grouted joint in testing, two cylinders which were tested at the age of 123 days failed in both the joint and the concrete. Figures 9 and 10 show cores before and after testing.

A later series of tests was made using cores drilled from blocks which were made with plane joints, keyed joints, and no joints. The joint width was set at 0.05-inch as before and grouted using pressures of 10, 30, and 50 psi. When these cores were tested, the grouted joints showed surprising strengths. Based on comparison with control cores drilled from blocks without joints, the efficiency of the grouted joints ranged from 81 to 125 percent. Typical results of the tests in this series are shown in Figures 11 and 12.

Subsequent to these studies, some tests were made of cores drilled across the contraction joints at Marshall Ford Dam. Although Marshall Ford Dam is a straight gravity dam, the same methods were used in grouting the contraction joints as for arch dams. After this dam had been completed and placed in service, a few drainage holes, BX-size, were drilled along the longitudinal contraction joint through the mass concrete. These holes were core-drilled and a few of the cores were found to contain excellent examples of grout film. At the time of drilling, no thought was given to making any tests of the grout film, and the cores were discarded by dumping them in a hole eroded by the side of one of the service roads. The cores were exposed to the weather for 2 or 3 years before they were noticed. The cores containing grout films were salvaged along with some check specimens for control and tested. These cores are shown in Figures 13 and 14.

Most of these cores were too short for standard test specimens. Three, however, were prepared for compressive strength tests by sawing into test lengths. Cores having grouted joints were cut so that the diagonal grout seam was entirely within the test length and the ends extended about 2 inches beyond the points where the grout seam intersected the sides of the specimen. This resulted in an average length of 6-1/2 inches for these specimens, thus producing a height-diameter ratio of approximately 3.8. Control cores without grout joints were sawed to the same dimension. The three cores with joints failed along the joint under an average compressive strength of 5,226 psi. Four control specimens failed under an average compressive strength of 6,140 psi.

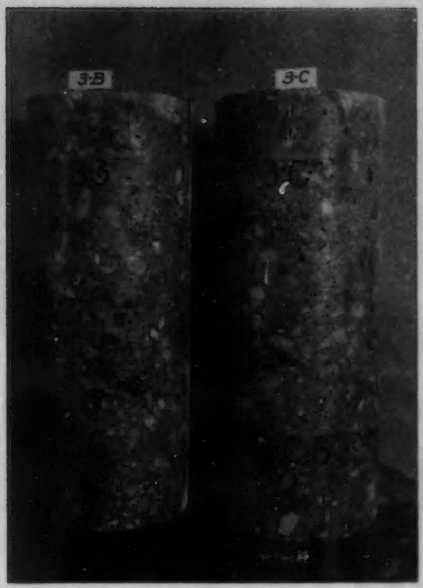


Fig. 9 Grouted Cores From Block No. 3 Before Testing

Fig. 10 Grouted Cores From Block No. 3 After Testing

Fig. 11 Core A-1 with Keyed Joint Showing Grout Layer Before Testing. Grouted at 50 psi.

Fig. 12 Core A-1 After Testing Showing Typical Fracture Where Specimen Failed Outside of Joint. Strength 4990.

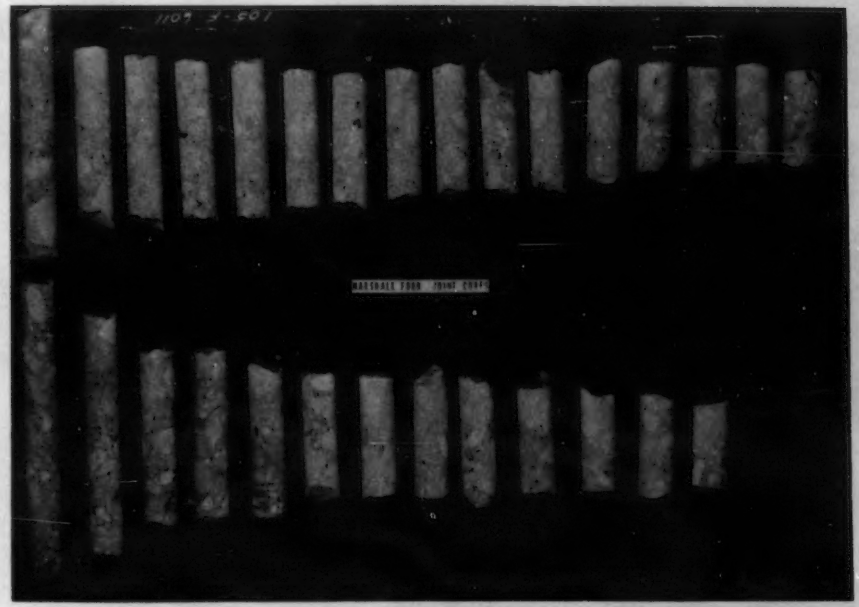


Fig. 13 General View of BX-size Cores Received From Marshall Ford Dam

Fig. 14 BX-size Cores From Marshall Ford Dam
Showing Grout Films

CONCLUSION

The effectiveness of grouting contraction joints of many arch dams has been investigated by drilling cores from across grouted joints. Many cores have been obtained which have excellent films of grout well-bonded to the adjacent concrete. These films have ranged in thickness from 1/50 inch to 3/8 inch. As the result of these investigations, it can be concluded that the system of constructing arch dams with contraction joints which are grouted with neat cement grout at the time the concrete of the dam is at its minimum volume, as now presently used by the Bureau of Reclamation, is an effective means of improving arch action in dams, minimizing cracking and reducing the possibility of leakage through the joints.

Paper 992 PO 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

"ARCH DAMS: SANTA GIUSTINA SINGLE-CURVATURE ARCH DAM"

Claudio Marcello,[1] M. ASCE
(Proc. Paper 992)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds: "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Gustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

This paper outlines the design and construction of an Italian arch dam 152 meters high and located in an extremely narrow gorge. Briefly discussed are

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1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

the structural analysis of the dam—confirmed by model, construction plant, and the extensive foundation consolidation and cut-off grouting.

The dam was built during the years 1946-1950 to provide a large seasonal reservoir with storage capacity of 183 million cubic metres in the Valley of Noce River (Trento) (Fig. 1).

The reservoir is intended for annual regulation of the Taio power station, which is directly fed from the reservoir, as well as for regulation of the plant of Mezzocorona, situated farther downstream, both plants being included in the Noce hydroelectric scheme and owned by Soc. Edison.

The first plan of the structure is to be attributed to the late Ing. Angelo Omodeo.

The final design was completed in 1943, although the construction work cound be actually started only in 1946. The time required for construction was 4 years, because of some difficulties connected with post-war conditions.

The dam has a maximum height of 152.50 metres above foundations, a length at crest of 124.20 metres, and a thickness at the crown varying from 3.50 m at the top to 16.50 m at the bottom. Maximum storage level is at El. 530 metres above sea level.

The structure is a single-curvature arch dam, in accordance with the shape of the gorge which has practically vertical walls: the arches are symmetrical, with thickness increasing from crown to abutments (Fig. 2).

The extrados of the elemental arches is circular with single center, the intrados is also circular, but with five centers and three radii, one of which corresponds to the middle center and the other two correspond to the two symmetrical pairs of center on either side (Fig. 3).

The dam is abutted directly on the rock up to the height of 102.50 m above foundations, on a lightly reinforced shoulder from the height of 102.50 to 142.50 m, and on solid gravity thrust blocks from this height to the crest.

Angle amplitudes are variable from 77°50' at the bottom to 108° at the top, the mean curvature radius increasing from 25.50 to 44.50 m.

Both dam facings are of bare concrete, and steel forms were used. No special plaster was applied for the lower portion of the upstream face, where a mesh-rendering is provided up to El. 446.

A light reinforcement was provided behind the facings for stress spreading purposes.

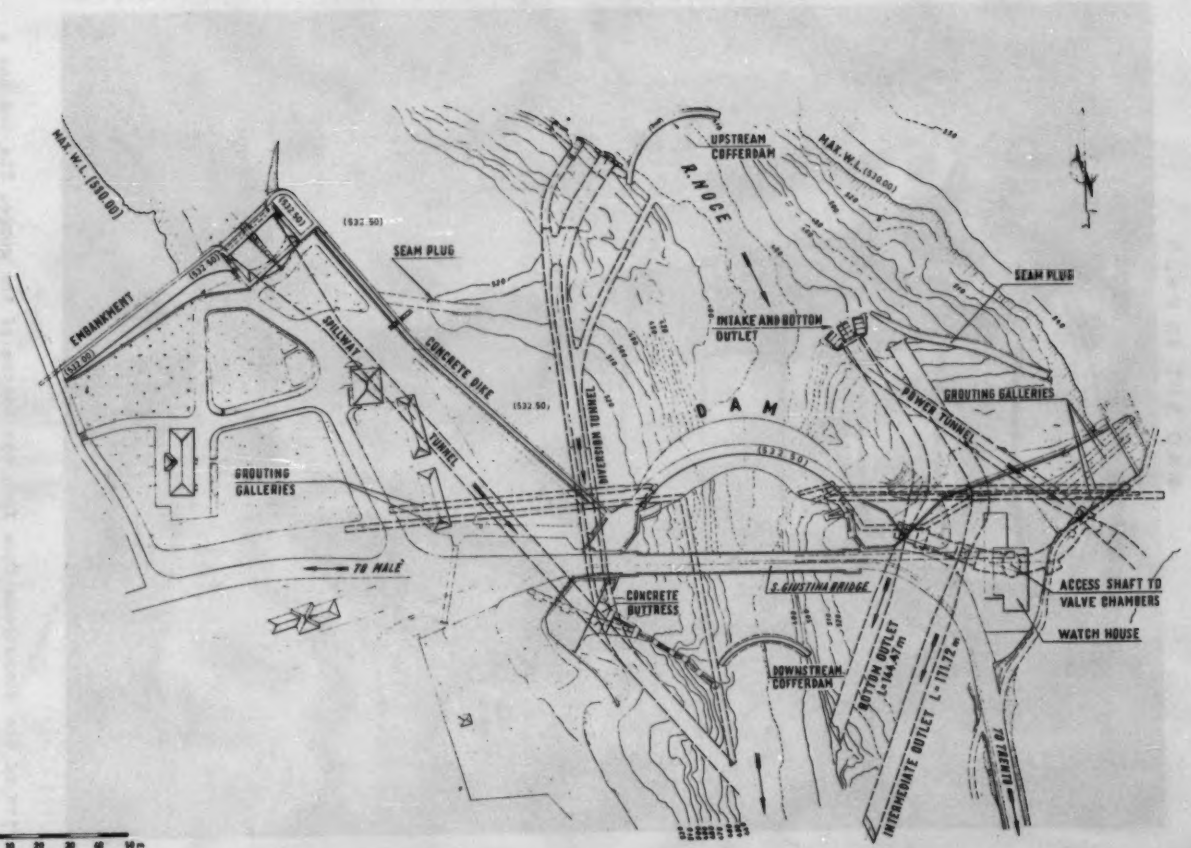
During construction the dam was divided into 7 blocks by 6 vertical temporary joints spaced from 12.50 to 15.50 m (measured on the upstream facing). The four joints in the central part divide the whole height of the dam, while the two lateral joints were provided only for the portion above El. 460 (Fig. 4).

After shrinkage was almost completed the joints were filled with concrete and subsequently cement grouted. Grouting was made by sections 15 m high, through a system of pipes embedded into the concrete, under low-temperature conditions (Fig. 6).

Grout leakage was avoided by the use of copper strips embedded by one half into the concrete block and one half into the joint fill.

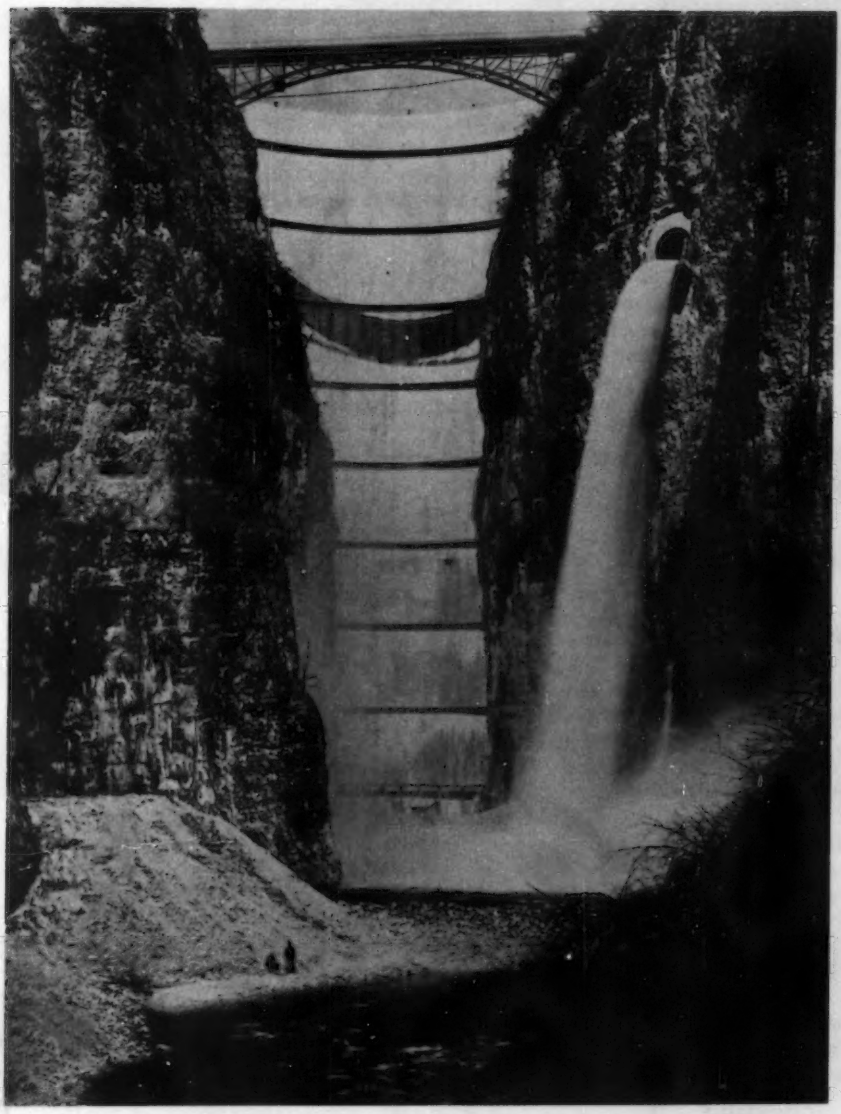
No special drainage system was provided inside the structure, whereas the foundations are drained by means of porous concrete pipes. These lead to a vertical shaft driven to the deepest point of a longitudinal scar in the bedrock

GENERAL LAYOUT



ASCE MARCELLO 992-3

FIG. 1



View of the downstream face from the bottom of the gorge. The dam has a maximum height of 152.50 m, a crest length of 124.20 m and a thickness at crown variable from 3.50 m at the crest to 16.50 at the bottom. See on the right the intermediate outlet under operation.

FIG. 3

PLAN OF THE DAM

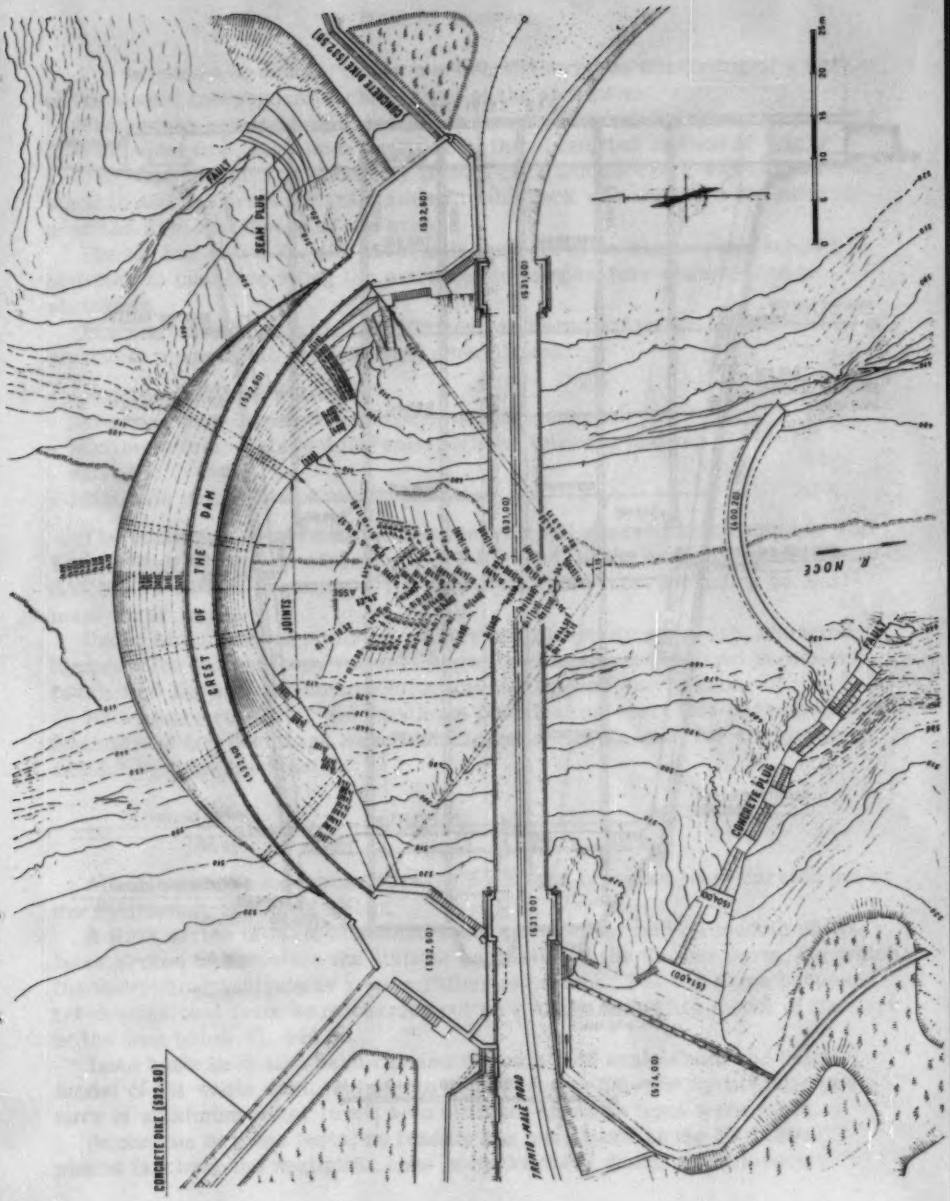
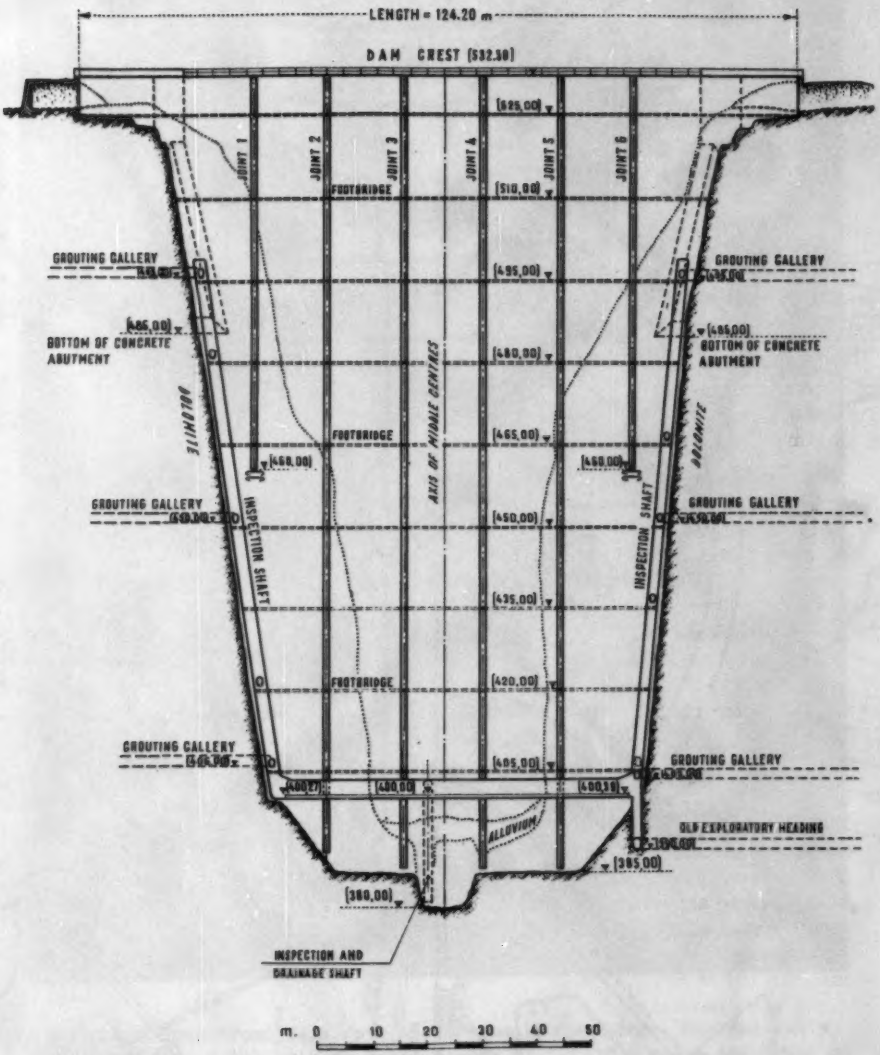


FIG. 4

SECTIONAL ELEVATION
DOWNSTREAM VIEW



and connected at its upper end with a horizontal inspection gallery about 20 m above the foundation plane. Access to this gallery is provided through two sloping adits running along the abutments.

Nine horizontal inspection footbridges spaced 15 m are arranged along the downstream face (Fig. 5).

Statical Analysis

For purposes of design, the dam was considered as consisting of a series of horizontal independent arches keyed at the abutments.

The arches below El. 460, for which the ratio radius/thickness is smaller than or equal to 3, have been analized by the simplified method of "rigid arches;" the modulus of elasticity of both rock and concrete was assumed as equal to and the pressure transmitted to the rock was assumed as uniform over the whole thickness of the arch.

The arches above El. 460 have been considered as elastic, and subject to hydrostatic pressure along the extrados, to temperature changes and to shrinkage.

The statical analysis of these arches has been performed by the method of elasticity ellipse with the following assumptions:

- Specific gravity of water 1.00 ton/m^3
- Shrinkage made equal to a temperature decrease of 3°C
- Maximum temperature difference between intrados and extrados 3°C
- Maximum yearly temperature variation 20°C

The maximum compression stresses with full reservoir amounted to 43.7 Kg/cm^2 and the tensile stresses to 3.9 Kg/cm^2, in the assumption of sole action of hydrostatic pressure at its maximum value (corresponding to maximum water level).

Under simultaneous action of maximum hydrostatic pressure, expected temperature fluctuations, and shrinkage, the maximum principal stresses reach 46.6 Kg/cm^2 for compression and 7.8 Kg/cm^2 for tension.

With empty reservoir the maximum principal stresses due to temperature fluctuations and shrinkage were found to reach 9.1 Kg/cm^2 for compression and 9.7 Kg/cm^2 for tension.

Model Testing

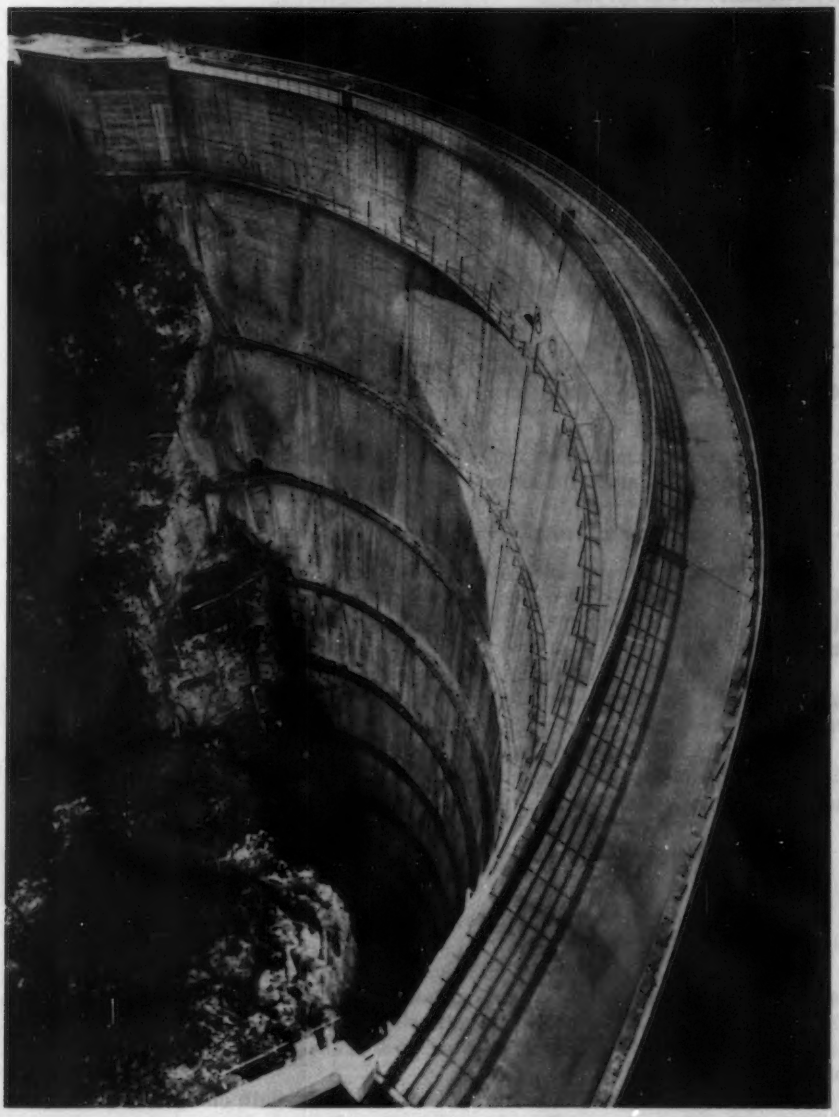
All calculations have been checked by means of model tests carried out at the Politechnic School of Milan.

A first series of bi-dimensional tests were performed on models of isolated arches to ascertain the statical behaviour of the thicker parts, for which the theoretical analysis is always rather uncertain. For the same purpose, tri-dimensional tests were carried out on a chalk and celite model of the part of the dam below El. 440.00.

Load tests have also been carried out on a 1:60 scale chalk and celite model of the whole dam, taking into consideration only the hydrostatic pressure at maximum water level; also ultimate strength tests were made.

According to these tests, no tension has been found in the horizontal planes (arches) and negligible ones in the vertical planes (cantilevers):

SANTA GIUSTINA DAM FIG.5

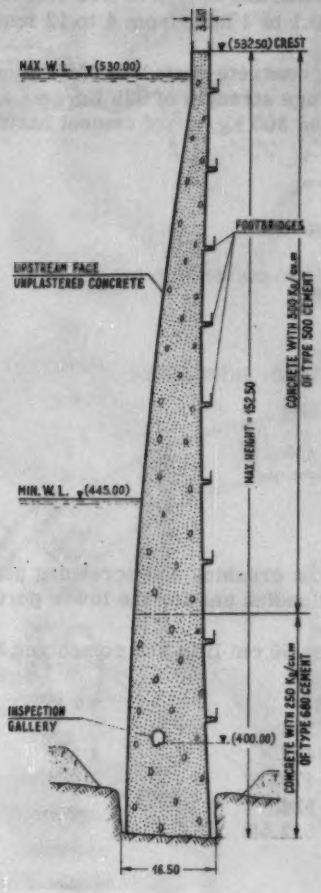


View from the left bank; see on the downstream face the horizontal inspection footbridges spaced 15 m.

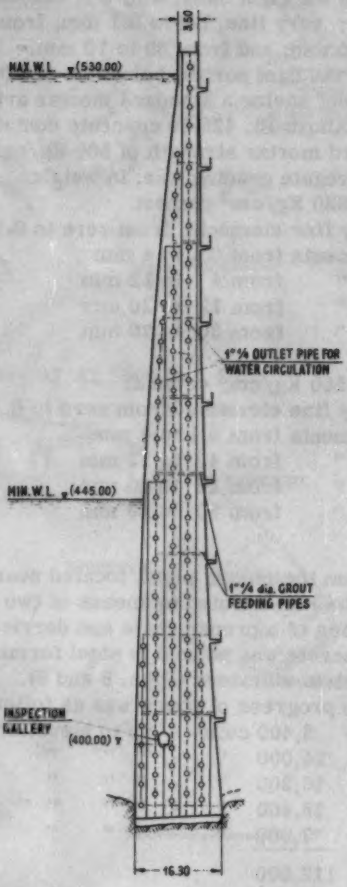
FIG. 6

CROSS SECTIONS

ALONG THE AXIS OF MIDDLE CENTRES



ALONG JOINT No. 4



compression is generally less than calculated, especially for the lower arches, except for a few regions where compressive stresses were found to be up to 10% higher than calculated.

Construction

Construction work was actually started in 1946; up to that time the only work performed concerned excavation, and preparation of quarrying, crushing and screening installations for aggregates.

Aggregates for the dam were obtained from a quarry of dolomite rock opened on the left bank, about 200 m upstream from the dam.

The rock was transported by cableway from the quarry to the crushing plant on the right bank, where it was crushed and sorted in five classes, namely: very fine, below 0.1 mm; from 0.1 to 4 mm; from 4 to 12 mm; from 12 to 30 mm; and from 30 to 70 mm.

For the dam portion below El. 425.00, concrete contained 250 Kg per m^3 of cement having a standard mortar average strength of 680 Kg/cm^2 after 28 days. Above El. 425.00 concrete contained 300 Kg/m^3 of cement having a standard mortar strength of 500 Kg/cm^2.

Aggregate grading was, in weight:

- with 680 Kg/cm^2 cement

very fine elements, from zero to 0.1 mm	12%
elements from 0.1 to 4 mm	22%
" from 4 to 12 mm	15%
" from 12 to 30 mm	21%
" from 30 to 70 mm	30%
	100%

- with 500 Kg/cm^2 cement:

very fine elements, from zero to 0.1 mm	12%
elements from 0.1 to 4 mm	22%
" from 4 to 12 mm	15%
" from 12 to 30 mm	25%
" from 30 to 70 mm	26%
	100%

From the mixing plant, located near the crushing and screening plant, concrete was distributed by means of two Blondins and for the lower portion also by means of a proper chute and derrick.

Concrete was poured in steel forms by 50 cm lifts and compacted by means of electric vibrators (Figs. 8 and 9).

The progress of pours was as follows:

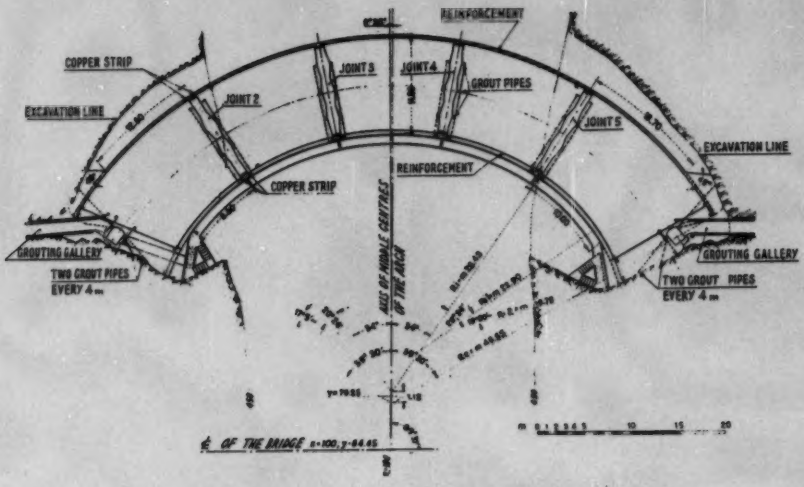
1946 -	6,400 cu.m	- up to elevation	396
1947 -	24,000	" - " " "	422
1948 -	46,200	" - " " "	479
1949 -	28,400	" - " " "	524
1950 -	7,000	" - " " "	532.50
Total	112,000		

When resuming the work after winter suspensions, the whole surface of the blocks as well as the bottom of points were prepared for the new pours by adequate cleaning and chipping to an average depth of 5 cm.

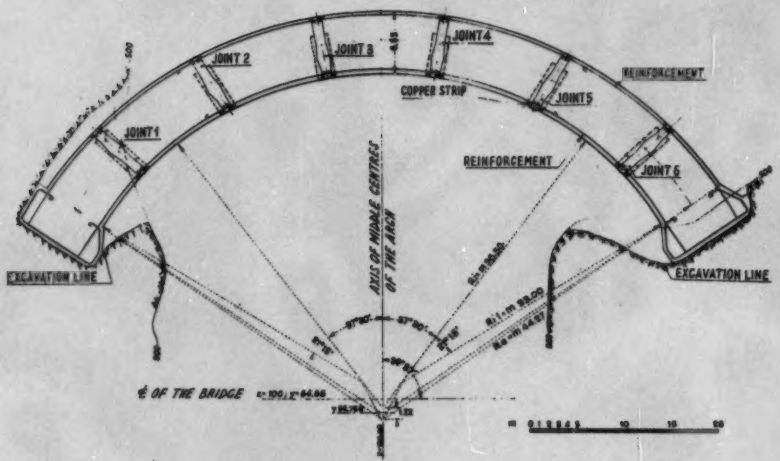
The filling of the joints left open during the preceding working season was

FIG. 7

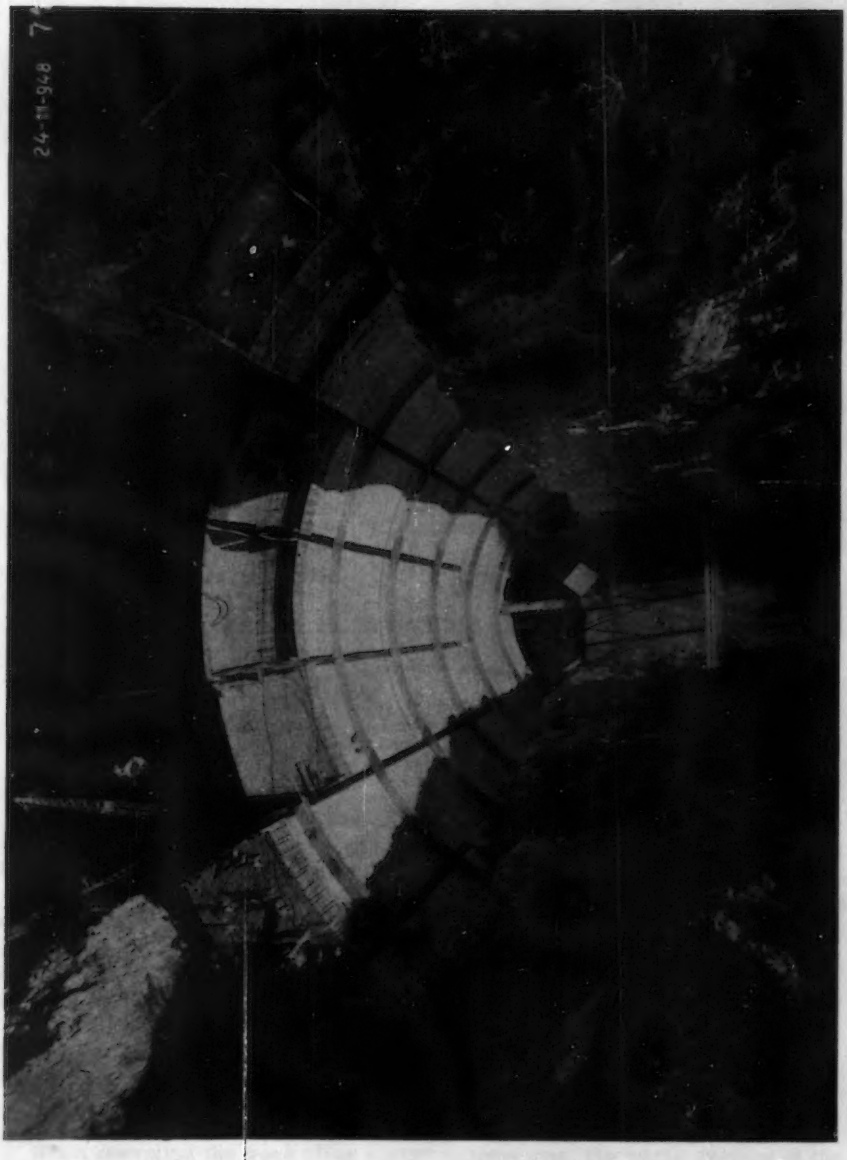
HORIZONTAL SECTION AT EL. 450.00



HORIZONTAL SECTION AT EL. 500.00

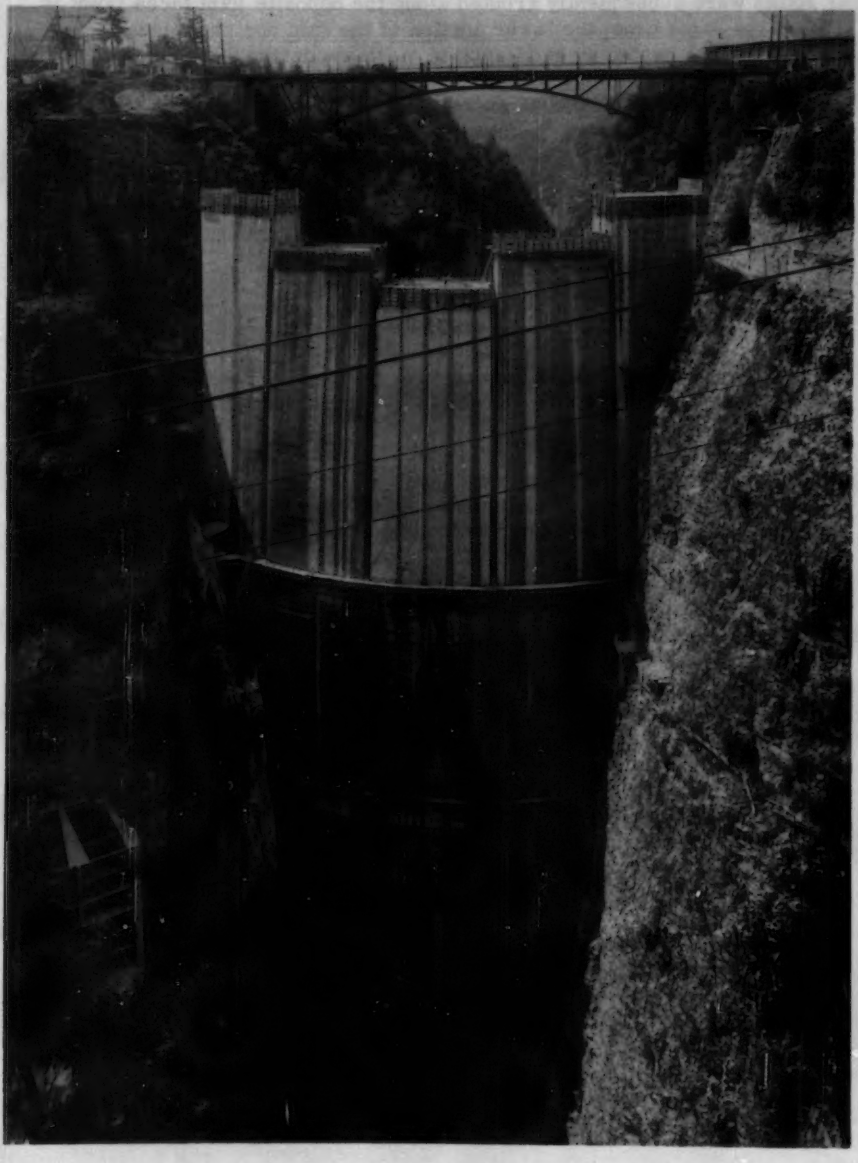


SANTA GIUSTINA DAM FIG.8



View of the dam under construction. The concrete pours were made by blocks divided by provisional construction joints to be filled with concrete and then to be grouted after shrinkage.

SANTA GIUSTINA DAM FIG.9



View of the upstream face when the construction was practically ultimated. The provisional joints spaced 12.50 ÷ 15.50 m are already closed and sealed in the lower part of the dam. See at the top the steel forms for the pours.

performed at the beginning of the subsequent one and concrete was used which had the same composition as the concrete of the adjacent blocks, so far as aggregate grading, and water and cement proportioning were concerned.

Joints were subsequently grouted along the opposite joint faces (joint to fill); at the same time, the lower portion of the dam upstream face (below El. 446.00) was coated by hand-applied mesh-rendering after chipping of the concrete.

Laboratory tests showed an average compressive strength of 420 Kg/cm^2 after 28 days for the concrete mixed with 250 Kg/m^3 of 680 Kg/cm^2 cement and 380 Kg/cm^2 for the concrete mixed with 300 Kg/m^3 of 500 Kg/cm^2 cement.

Average strength after 1 year was for both types higher than 470 Kg/cm^2 with peaks of 700 Kg/cm^2.

The concrete was found to be practically impervious.

Waterproofing

Waterproofing work had a considerable importance because the dolomite rock on which the dam is founded showed horizontal stratifications with faults and a number of small jointing (Fig. 10).

A deep grout curtain had therefore to be provided, with holes driven down to 50-60 m below foundation plane.

Three tunnels were driven into the gorge walls at Els. 406 - 450 and 495, reaching about 100 m into the rock. Vertical grout holes were drilled downwards from these tunnels, spaced 2.50 m centers to a depth of over 50 m, so as to cover the whole underlying rock section down to the next tunnel.

Sloping grout holes were also drilled from the abutments blocks, from the above said grouting tunnels and from the outlet tunnels, in order to obtain a good connection between the vertical groutings. The holes, driven by rotary drill, were grouted by sections at a pressure of 80 atmospheres by means of compressed air driven pumps and hydraulic leakage tests were made at pressures of 10 to 12 atm.

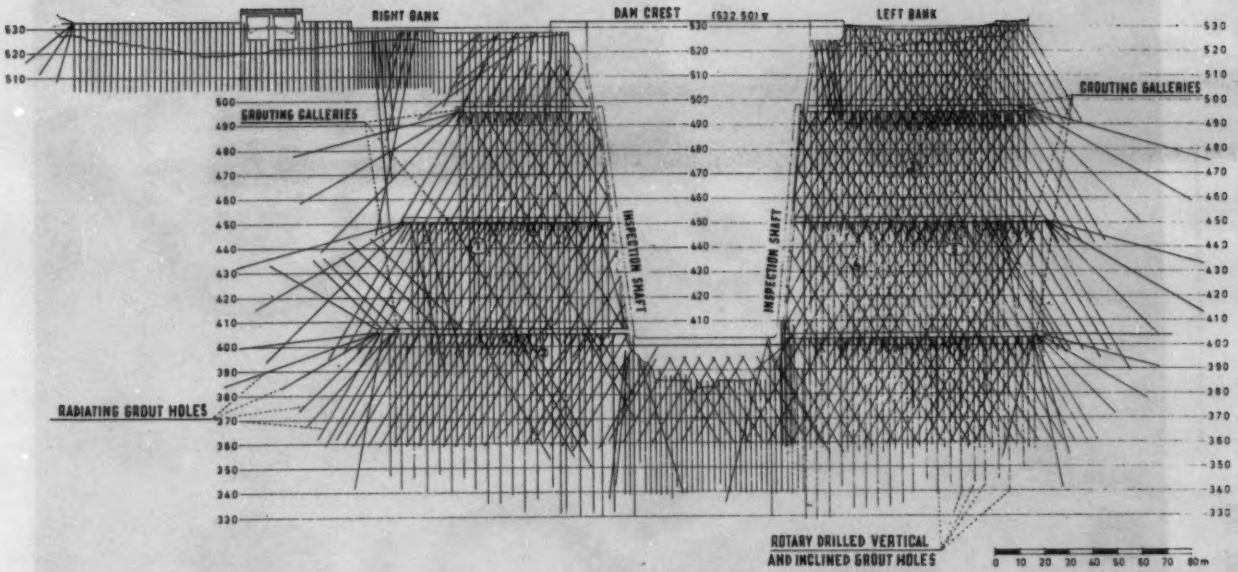
The water-cement ratio of the mix used for grouting varied between 8 to 1 and 6 to 1. A total of over 65,000 m of holes were drilled: average absorption was little more than 100 Kg. of cement per metre of bore.

The faults and seams which endangered watertightness were filled with concrete plugs. A seam crossing the gorge upstream of the dam and extending on both banks was plugged with concrete and grouted after accurate cleaning and washing.

Many precautions had to be adopted for the right bank, where a fault could have affected the stability of the structure; this fault was subjected to a thorough washing by means of high-pressure jets (up to 30 atm) driven through a series of holes drilled so as to cross it, and was subsequently grouted. Some small cavities in the rock were filled with concrete and grouted.

Consolidation grouting at a pressure of over 70 atms. was carried out at the abutments, through 424 holes, and so distributed as to cover a mass of

GROUT CURTAIN
DOWNSTREAM VIEW

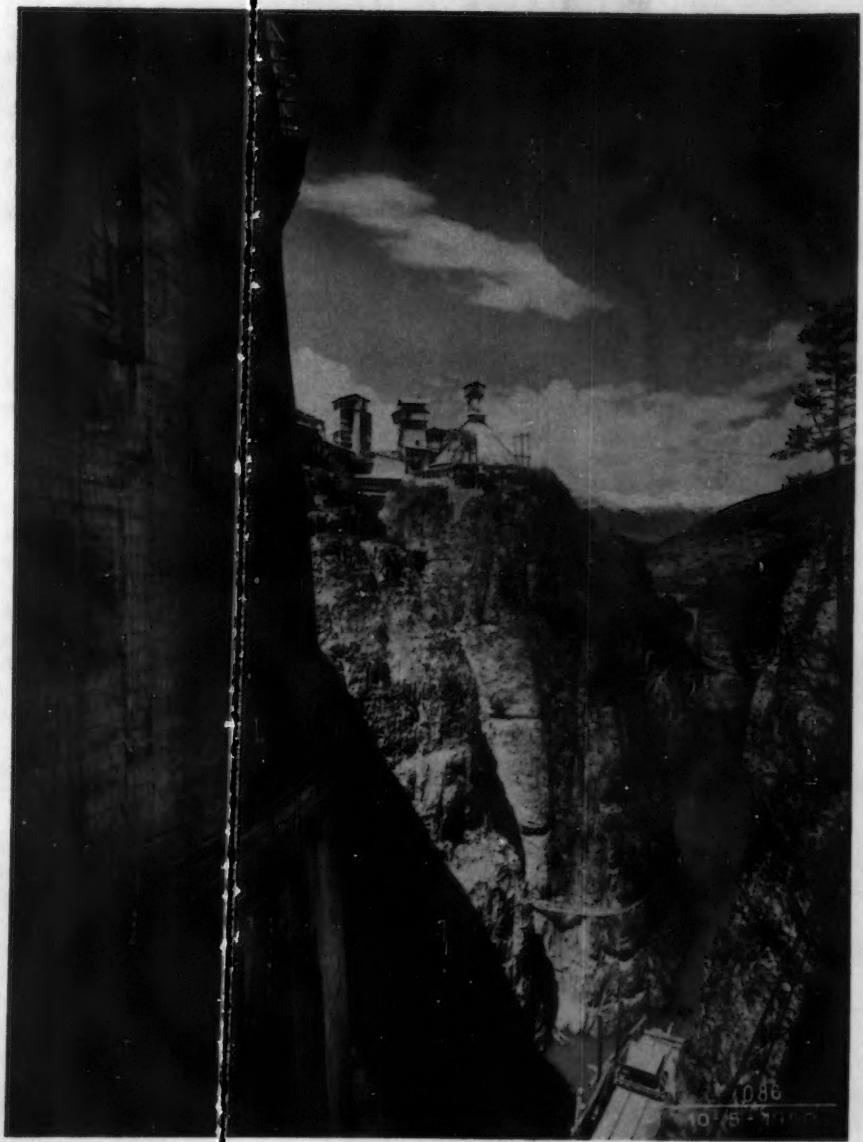


1 SPILLWAY TUNNEL
2 DIVERSION TUNNEL
3 INTERMEDIATE OUTLET
4 BOTTOM OUTLET
5 POWER TUNNEL

TOTAL LENGTH OF GROUT HOLES 65,000 m
GROUTING AT MAX PRESSURE OF 80 Kg/sq. cm;
CEMENT/WATER RATIO OF GROUT VARYING FROM 8:1 TO 0:1
INJECTED CEMENT: 0,1 METRIC TONS PER METRE OF GROUT HOLE

FIG. 10

SANTA GIUSTINA DAM FIG. 11



Detail of the upstream face. In the background the field installations.

rock sufficient to support by itself the thrust of the dam. The total length of the holes grouted is 18.424 m with absorption of 1,020.5 tons of cement.

The dam was designed and built by the Hydroelectric Plant Construction Department of Soc. Edison of Milan, under the direction of the writer.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: MEASUREMENTS AND STUDIES ON SANTA GIUSTINA DAM

Claudio Marcello,[1] M. ASCE
(Proc. Paper 993)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 993 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

SYNOPSIS

Owing to the unusual proportions of this structure, special plans were made to study its behavior resulting from variations of water level and temperature. This paper is a preliminary report and presents data obtained from four years of observation and compares these results with those obtained by analysis.

This paper presents some preliminary results of an investigation concerning Santa Giustina Dam, a dam built in Northern Italy after World War II.

The dam spans a very deep gorge with extremely steep walls, on the course of Noce River, and forms a reservoir with a storage capacity of 183,000,000 m^3. The dam is a practical symmetrical, 152.50 m high, thin-arched structure. The thickness in the crown section varies from 3.50 m at the crest (el. 532.50 m a.s.l.) to 16.50 m at the foot (el. 380 m a.s.l.); the chord at the crest is 75 m approx.; the upstream arches have curvature radii variable from 49.15 to 39.35 m and span angles varying from 106° to 74° (see Figs. 1 and 2).*

Owing to the uncommon dimensions of this structure, it was considered particularly interesting to investigate its behaviour in respect to the variations of water level and temperature by direct measurement, and to compare these results with those obtained by various methods used for the analysis of arch dams.

To obtain the required experimental data, the dam was provided with a system of thermometers and a system of extensometers. The periodic reading of these instruments (daily temperature readings and weekly strain readings) was supplemented by systematic geodetical measurements. These latter consisted mainly of the measurement, once every three months approx., of a triangulation intended to give the planimetric coordinates of a series of points of the downstream face of the dam. Geometrical levellings were also carried out, to give the elevations of a series of points of the dam as well as of the gorge walls.

This first report takes into consideration the radial deflections of points of the central cantilever, as measured in the first 20 triangulations made during as long as four and a half years, and the strains and stress increases found by extensometrical measurements carried out at the same time as the geodetical measurements.

As far as theoretical analysis is concerned, the paper gives the first results obtained by two methods intended to calculate the deflections and the stress variations in the dam in function of temperature and water level variations occurring between the first geodetical measurement and those following. These results are then compared with the data already found by experience.

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*A detailed description of the dam is the subject of another paper entitled "Santa Giustina Single-Curvature Arch Dam," illustrated in this same Symposium.

SANTA GIUSTINA DAM

FIGURE 1- CROSS SECTION OF THE DAM
AT CROWN CANTILEVER — SCALE 1:1000

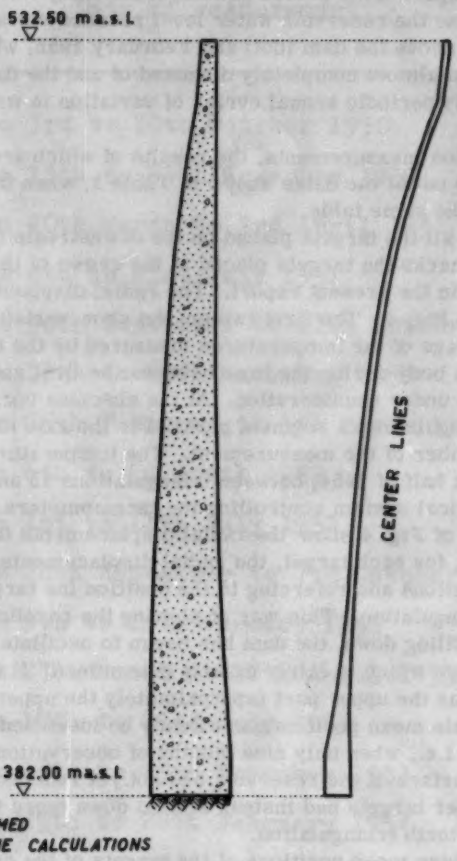
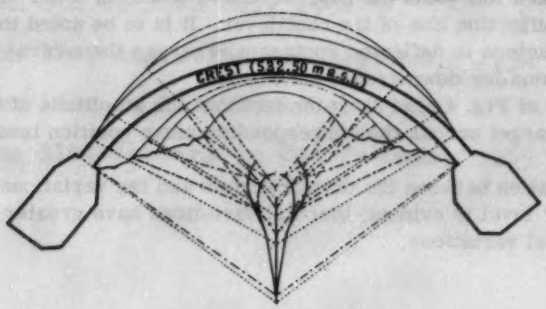


FIGURE 2- PLAN OF THE DAM - SCALE 1:1000



Geodetical Measurements—Deflections of the Central Cantilever

Out of all the results of the geodetical operations carried out to determine the movements of the Santa Giustina dam, only the deflections of the central cantilever measured during four and a half years of investigations are reported in this first paper. This period extends from early October 1950, when for the first time the reservoir water level reached elevation 435 m a.s.l. (equal to 53 m above the dam foot) and February 1955, when the setting heat had already been almost completely disposed of and the dam had undergone three practically periodic annual cycles of variation in water level and temperature.

The 20 triangulation measurements, the results of which are here reported, were carried out at the dates shown in Table 1, when the water level was as indicated in the same table.

Fig. 3 illustrates all the targets placed on the downstream face of the dam; a filled-in triangle marks the targets placed at the crown of the arches, i.e., the ones considered in the present report. The radial displacements of these targets are shown in Fig. 4. The first two graphs show variations in water level and in the average of the temperatures measured by the thermometers embedded in the dam body during the time between the first and the last of the 20 triangulations under consideration. At the abscissa corresponding to the date of each triangulation, a segment parallel to the axis of ordinate shows the serial number of the measurement. The temperature graph is interrupted in the first half of 1954, between triangulations 15 and 16, owing to a break in the electrical system controlling the thermometers.

The other graphs of Fig. 4 show the radial displacements of the targets. A broken line shows, for each target, the radial displacements resulting from the various triangulations and referring to the position the target had at the time of the first triangulation. This way of plotting the results makes it clear that, after a first settling down, the dam has begun to oscillate across a mean position of equilibrium which is fairly exactly determined. It is to be remarked that, as far as the upper part (approximately the upper third) of the dam is concerned, this mean position can already be identified after the fourth triangulation, i.e., when only nine months of observations had been made and the free surface of the reservoir had not yet reached El. 500 m a.s.l. The three lower targets had instead settled down more than a year later, i.e., after the tenth triangulation.

The distance of these mean positions of the targets of the central cantilever from the positions determined at the time of the first triangulation is shown in Fig. 5. A broken line joins the representative points in order to show the approximately deflection line of the cantilever. It is to be noted that the upper half of the structure is deflected upstream, whereas the central part of the dam shows a smaller downstream deflection.

The graphs of Fig. 4 also show the fact that the amplitude of the oscillations of each target across the corresponding mean position tends to decrease with time.

The correlation between the displacements and the variations of temperature and water level is evident; thermal variations have greater influence than water level variations.

Table 1

Triangulation Measurements

Measurements	Date of measurement	Mean water level [m a.s.l.]
1st	from 3rd to 10th October 1950	435.49
2nd	from 15th to 25th November 1950	474.20
3rd	from 20th March to 3rd April 1951	500.22
4th	from 8th to 21st July 1951	497.62
5th	from 30th September to 11th October 1951	522.99
6th	from 15th to 25th November 1951	529.96
7th	from 26th February to 5th March 1952	492.28
8th	from 6th to 20 April 1952	484.11
9th	from 1st to 11th July 1952	525.43
10th	from 28th September to 7th October 1952	528.82
11th	from 9th to 19th January 1953	505.81
12th	from 27th March to 12th April 1953	475.65
13th	from 17th to 23rd July 1953	517.74
14th	from 18th to 25th November 1953	529.00
15th	from 25th to 28th February 1954	497.59
16th	from 29th April to 1st May 1954	473.27
17th	from 8th to 10th July 1954	527.80
18th	from 31st August to 2nd September 1954	527.09
19th	from 16th to 18th November 1954	509.67
20th	from 31st January to 2nd February 1955	501.74

SANTA GIUSTINA DAM

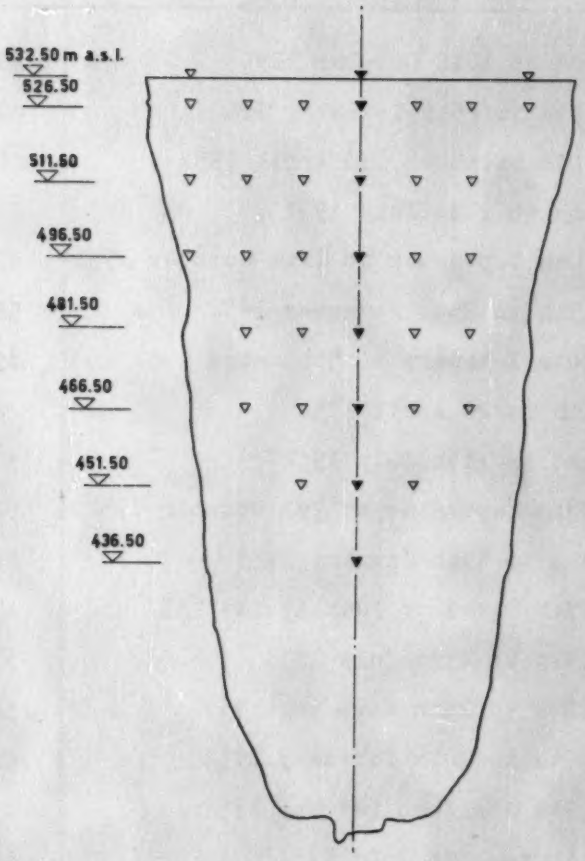


FIGURE 3 - TRIANGULATION TARGETS AT DOWNSTREAM FACE

Measurements with the Extensometers--Strains and Stresses

The Santa Giustina dam acts essentially as an arch dam: for this reason it was provided with a system of extensometers designed to control only one cantilever, the central one, and four arches, at Els. 406, 451, 481 and 526 m a.s.l. respectively.

The purpose of this installation was to measure the strains at the boundary of the sections at the crown and at the abutments.

To this end, twelve sets of attachments for removable extensometers were provided on the downstream face of the dam and they were specifically: one set at the crown and two sets at the abutments of each of the four arches in question. Each set of attachments allows for four positions of the extensometer: one vertical, one horizontal and two at 45° to the others. In the upstream side of the dam, at a distance of approx. 25 cm from the face, one horizontal electroacoustical extensometer was embedded in the concrete near to both abutments; one horizontal and one vertical extensometers were similarly embedded in the crown section.

Immediately to the left of the crown section of the four arches in question, two more sets of attachments for removable isolated extensometers were provided on the downstream face (these attachments allow for only horizontal and vertical positions); one isolated electroacoustical extensometer was similarly embedded vertically in the concrete near to the upstream facing. Each downstream set of attachments was placed in a region isolated from the rest of the structure by means of a peripheral groove; this groove was cut to a sufficient depth to prevent the transmission of stresses. The strains measured in these conditions are due only to local temperature variations, to shrinkage or swelling of concrete or to other causes, excluding however the stresses.

Strains due only to stresses are obtained by subtracting the strains measured by isolated extensometers from the strains measured by the other ones. The stresses may then be found by means of well known formulae of the Mechanics of Continua.

Obviously, the extensometers only allow the evaluation of the stress variations occurring between two subsequent measurements: the evaluation of the actual stresses existing when the second measurement is taken requires a knowledge of the stresses existing during the first measurement.

* * *

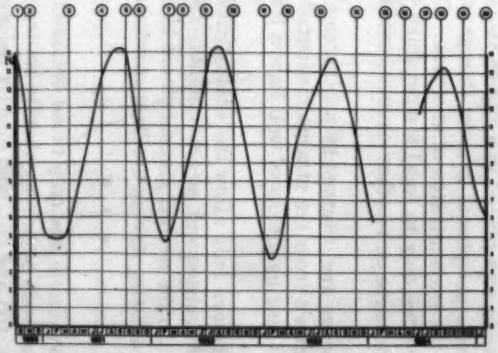
The strain values now available are only those obtained from the removable extensometers in the periods beginning shortly before the third triangulation, for the arches at Els. 406 and 461, and shortly before the fourth triangulation, for the arches at Els. 481 and 526.

These removable extensometers are mounted on a 800 mm base and have an amplification ratio equal to 250.

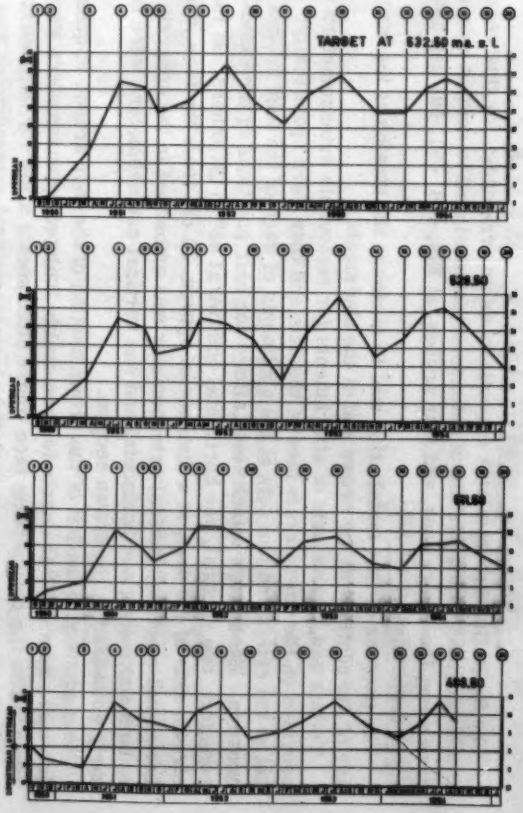
The results of the strain measurements have been elaborated with the purpose of determining the unit strains and the stress variations occurring between the first triangulation, for which extensometrical measurements are available, and subsequent ones. The following method was adopted.

As already said, readings on the extensometers are usually taken once a

AVERAGE TEMPERATURE OF THE DAM



RADIAL DISPLACEMENTS OF THE TARGETS PLACED AT THE CROWN OF THE ARCHES



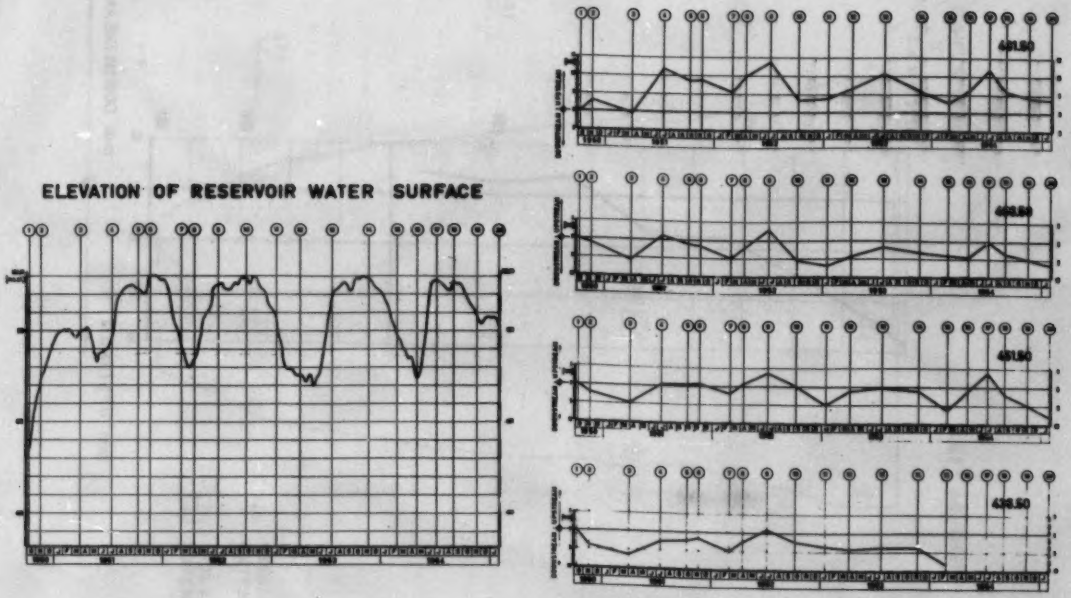


Fig. 4. Temperatures; water surface level; radial displacements of the targets placed at the crown of the arches.

SANTA GIUSTINA DAM

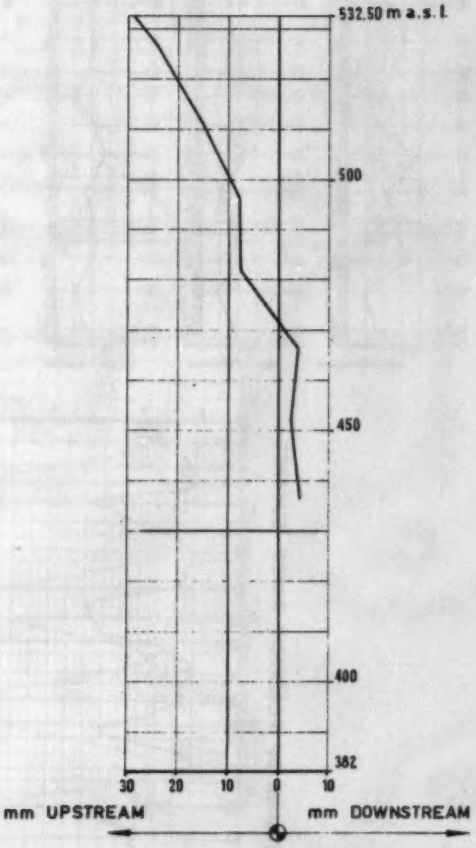


FIGURE 5 - AVERAGE RADIAL DISPLACEMENTS OF THE TARGETS PLACED AT THE CROWN OF THE ARCHES, WITH RESPECT TO THE POSITIONS RECORDED DURING THE FIRST MEASUREMENT

SANTA GIUSTINA DAM

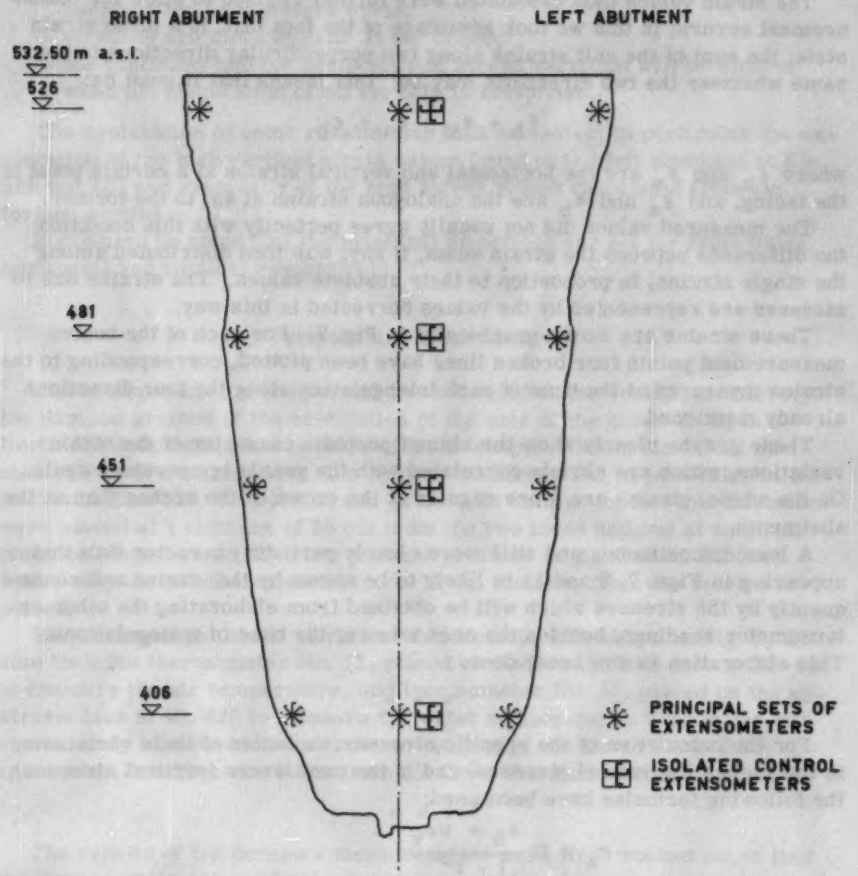


FIGURE 6 - REMOVABLE EXTENSOMETERS: MEASUREMENT POINTS

week. When, during one triangulation, more than one reading was taken, then their average was introduced into the calculations. The total strains detected by the extensometers were calculated as the difference between the values corresponding to each triangulation and the ones corresponding to the reference triangulation (triangulation No. 3 for the arches at El. 406 and El. 451, and triangulation No. 4 for the upper two arches). The average of the vertical and horizontal strains shown by the isolated extensometers placed in the same arch was subtracted from these total strains.

The strain values thus calculated were further revised to allow for measurement errors; in this we took advantage of the fact that, in a plane strain state, the sum of the unit strains along two perpendicular directions is the same whatever the two directions may be: this means that it must be:

$$\epsilon_h + \epsilon_v = \epsilon_a + \epsilon_b$$

where ϵ_h and ϵ_v are the horizontal and vertical strains at a certain point of the facing, and ϵ_a and ϵ_b are the analogous strains at 45° to the former.

The measured values did not usually agree perfectly with this condition: the difference between the strain sums, if any, was then distributed among the single strains, in proportion to their absolute values. The strains due to stresses are represented by the values corrected in this way.

These strains are shown graphically in Fig. 7. For each of the twelve measurement points four broken lines have been plotted, corresponding to the strains measured at the time of each triangulation along the four directions already mentioned.

These graphs clearly show the almost periodic character of the strain variations, which are closely correlated with the yearly temperature cycle. On the whole, strains are more regular at the crown of the arches than at the abutments.

A less discontinuous and still more clearly periodic character than that appearing in Figs. 7, 8 and 11 is likely to be shown by the strains and consequently by the stresses which will be obtained from elaborating the other extensometer readings, besides the ones taken at the time of traingulations. This elaboration is now being done.

* * *

For the calculation of the specific stresses, or better of their variations, in the arches (horizontal stresses) and in the cantilevers (vertical stresses), the following formulae have been used:

$$\sigma_a = E \frac{\epsilon_h + \nu \epsilon_v}{1 - \nu^2}$$

$$\sigma_c = E \frac{\epsilon_v + \nu \epsilon_h}{1 - \nu^2}$$

$$\tau = \frac{E}{2(1+\nu)} (2\epsilon_a - \epsilon_h - \epsilon_v),$$

where Young's modulus of elasticity of concrete E was assumed to be equal to 400,000 kg/cm^2, based on the results of measurements carried out by

means of a soniscope, and Poisson's ratio to be equal to 1/6.

The results are plotted in Fig. 8: this shows the progressive stress variation indicated by the extensometers, which is referred to the unknown stress values at the time of the third triangulation, for the two lower arches, and at the time of the fourth one, for the two arches at Els. 481 and 526 m.

Observations similar to those made on Fig. 7 for the strains can also be made on Fig. 8, and they are specifically:

- the stress variation shows an annual periodicity and is mainly influenced by the temperature cycle;
- stresses are more regular at the crown;
- the continuity and the periodicity of stress variation will be clearer when the elaboration of all the data is completed.

The explanation of some results may then be easier, in particular the explanation of the high vertical strain values found at the left abutment at Els. 526 and 481 and eventually of the high stress values calculated from the former strains.

It can also be noted that the principal directions are not far from being horizontal and vertical respectively.

Temperature Measurements

The measurement of temperature was limited to the central cantilever of the dam, on account of the orientation of the axis of the gorge being in a North-South direction and because of its U-shape. Eleven electrical thermometers were embedded in this central cantilever and in particular in the arches at Els. 527, 480, 455.50 and 407.50. In each arch, two thermometers were placed at a distance of 35 cm from the two faces and one at an equal distance from them: this latter was not provided in the arch at El. 506, where only the two thermometers at a 35 cm distance from the faces were placed.

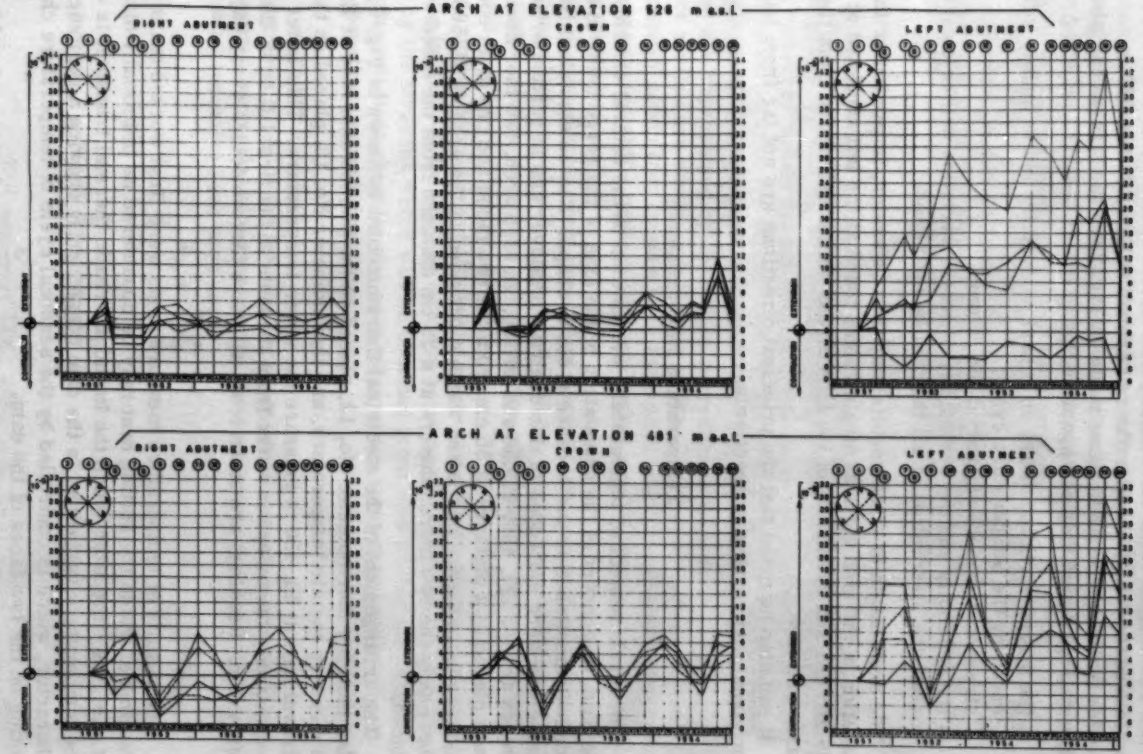
The arrangement of the electrical thermometers is shown in Fig. 9, which also includes thermometer No. 12, placed on the downstream face at El. 480 to measure the air temperature, and thermometer No. 13, placed on the upstream face at El. 430 to measure the water temperature. This latter was continuously immersed in water from the time of the first filling of the reservoir. Readings are taken from all these thermometers every day.

* * *

The results of temperature measurements were first worked out to find the time at which the setting heat may be considered as practically disposed of, i.e., the time after which the initial aperiodic thermal transient is over, the temperature variations in the dam consequently showing a periodical character, which is controlled by the seasonal cycle of temperature changes acting on the two faces of the dam.

The yearly moving averages of the temperature values measured by each of the thermometers embedded in the dam have been calculated with this purpose: this means that, month by month, the averages of the temperatures recorded by the same thermometer in the previous twelve months have been

SANTA GIUSTINA DAM



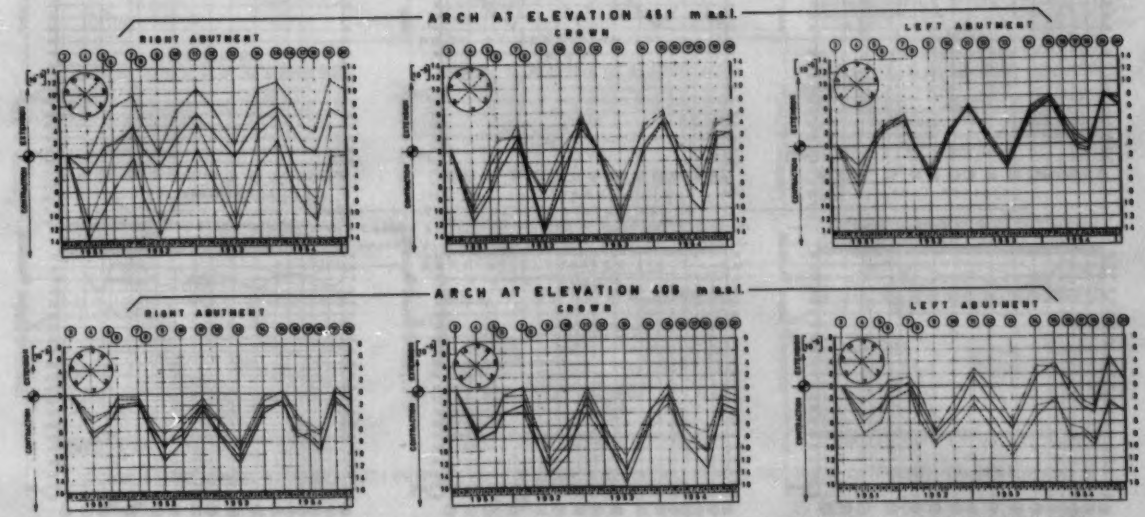
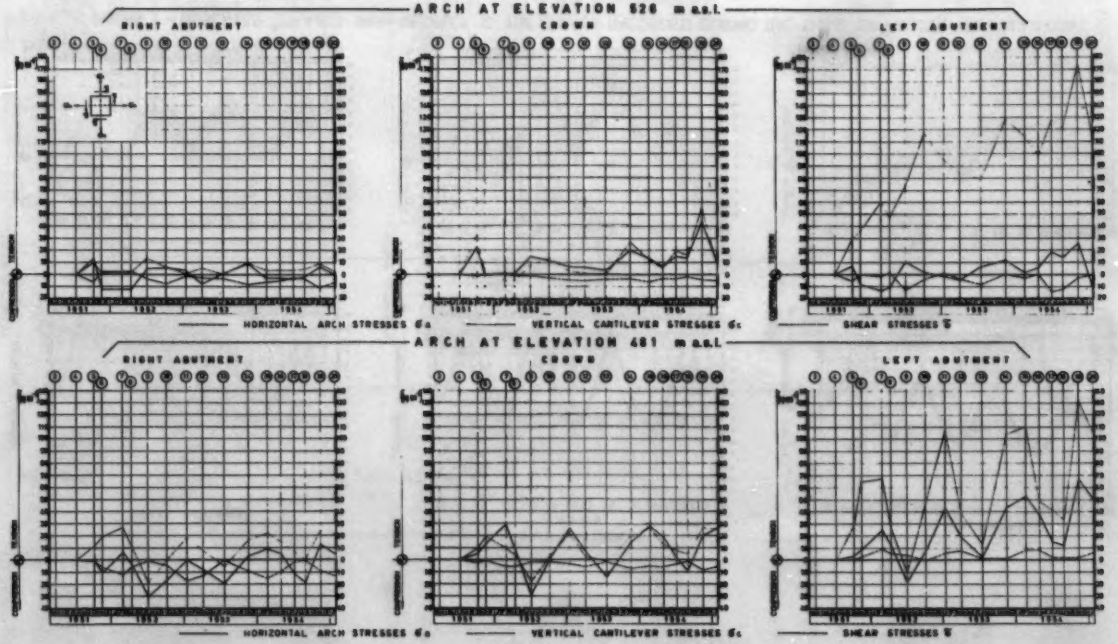


FIGURE 7 - OBSERVED STRAINS WITH RESPECT TO THE STATUS RECORDED DURING THE THIRD OR FOURTH TRIANGULATION

SANTA GIUSTINA DAM



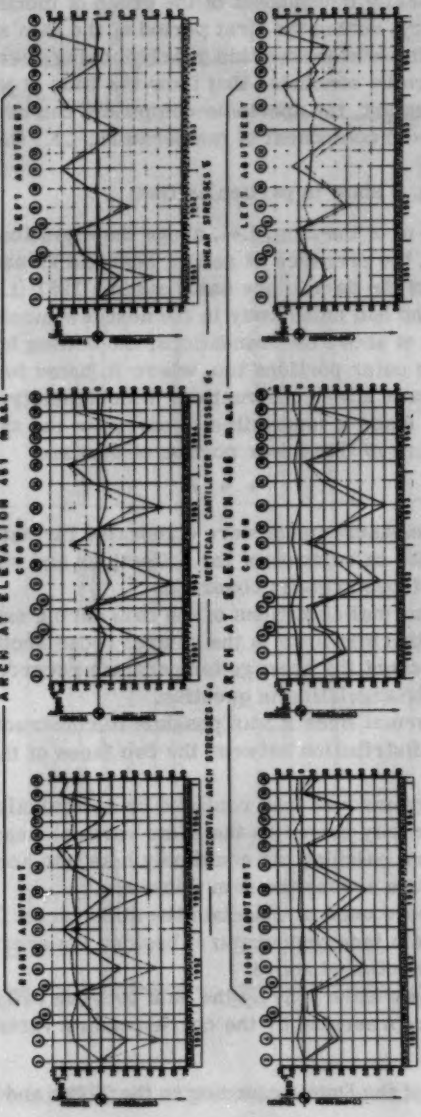


FIGURE 8 - OBSERVED STRESSES WITH RESPECT TO THE STATUS RECORDED DURING THE THIRD OR FOURTH TRIANGULATION

computed, obviously starting from one year after the beginning of observations. These averages are plotted in Fig. 9.

When the temperature measured by a thermometer is influenced only by the seasonal cycle of temperature changes, the average for twelve consecutive months is practically independent of the group of months to which it applies. On the contrary, during the first period of the dam's existence, the dispersal of the setting heat makes this yearly moving average decrease with time. It can therefore be concluded that from the time at which this moving average becomes constant, the aperiodic component can no longer be superimposed on the periodic component of temperature, i.e., the setting heat has no more influence.

The graphs of Fig. 9 show in particular that:

- in the upper 70 m of the dam, i.e., above the elevation of thermometers nos. 4, 5 and 6, the presence of setting heat had already been limited to the inner part of the dam, since early autumn 1951 (i.e., 2-3 years from construction) and had faded away in the next few months;
- in the lower 30 m above the foundations, the setting heat was still remarkable in the outer portions too, where it lasted for another year, i.e., for five years after construction. Unfortunately, a break in thermometer No. 2 made it impossible to carry out the same investigation for the inner core of this lower portion of the dam.

* * *

The temperature measurements were elaborated further as far as they were necessary for the calculation of the deflections and stresses, which were to be compared with those actually observed.

First of all, the thermal conditions of the dam during each triangulation were found. The isothermal lines in the central cross section were then drawn, taking into account the average temperature recorded by each thermometer during the triangulation in question.

From these isothermal lines it was possible to construct diagrams showing the temperature distribution between the two faces of the dam at 15 different elevations.

Each of these diagrams was then replaced by an equivalent trapezoidal diagram having its gravity center on the same vertical line; this was done because the subsequent calculations could only take into account a linear distribution of temperature across the dam thickness.

These diagrams were used to calculate the differences D between the upstream and downstream face temperature, besides the average temperature t at the 15 elevations mentioned above.

The thermal changes undergone by the dam between two triangulations were assumed to be represented by the corresponding variations of t and D.

Statical Analysis of the Dam According to the Tölke and Ritter Methods

So far calculations have been carried out, to find the deflections which might be expected at the arch crowns and the stresses at the strain measurement points at the time of each triangulation.

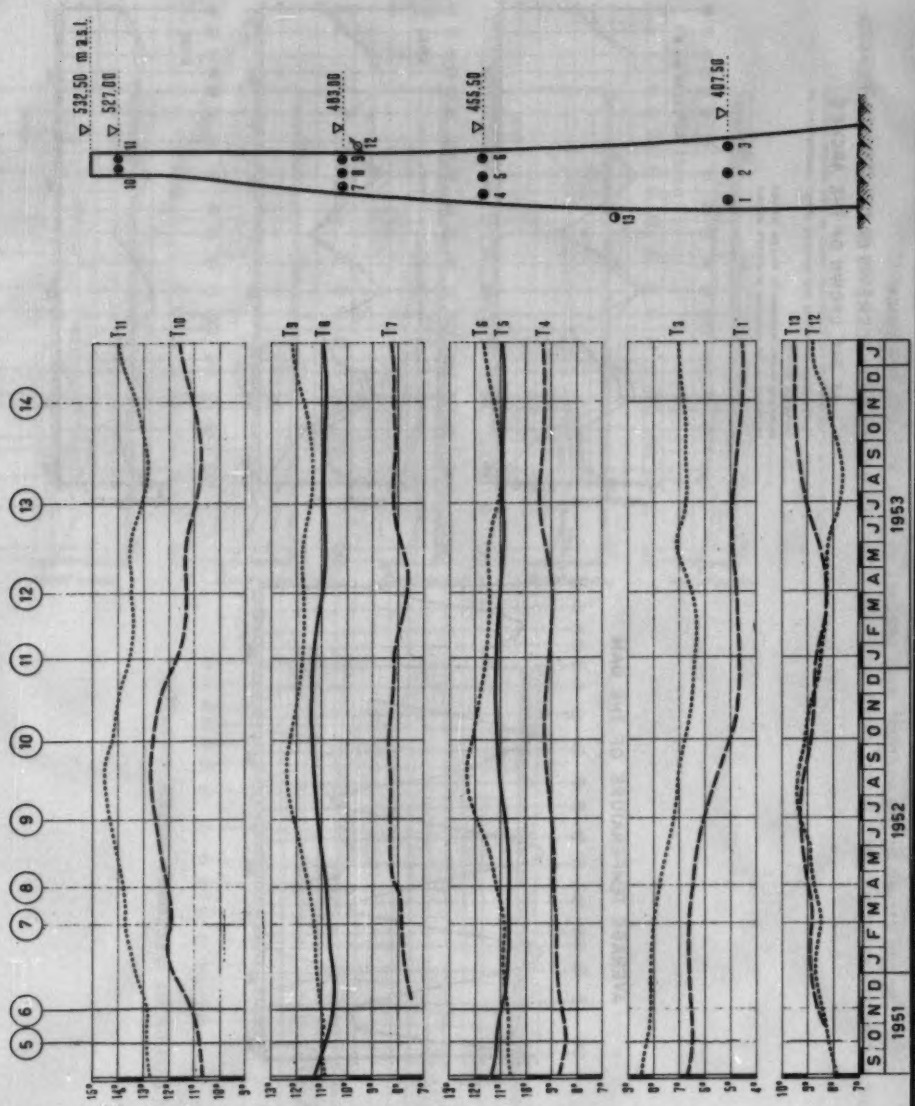
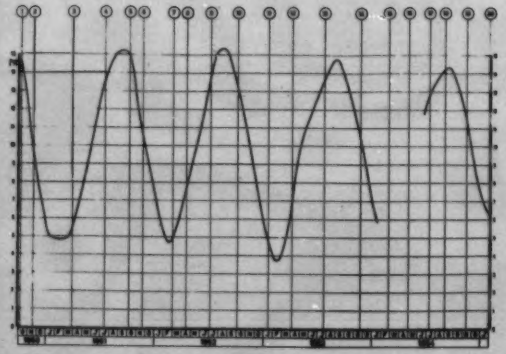


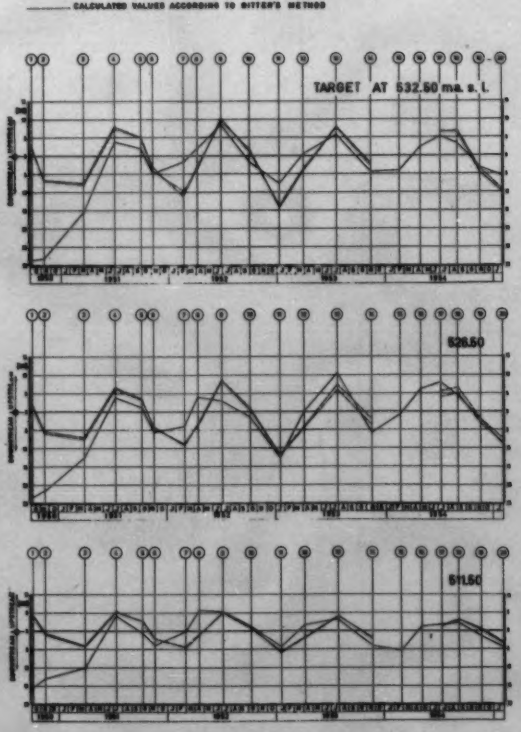
FIGURE 9

ARRANGEMENT OF THE ELECTRICAL THERMOMETERS; YEARLY MOVING AVERAGE OF RECORDED TEMPERATURES

AVERAGE TEMPERATURE OF THE DAM



RADIAL DISPLACEMENTS OF THE TARGETS PLACED AT THE CROWN OF THE ARCHES



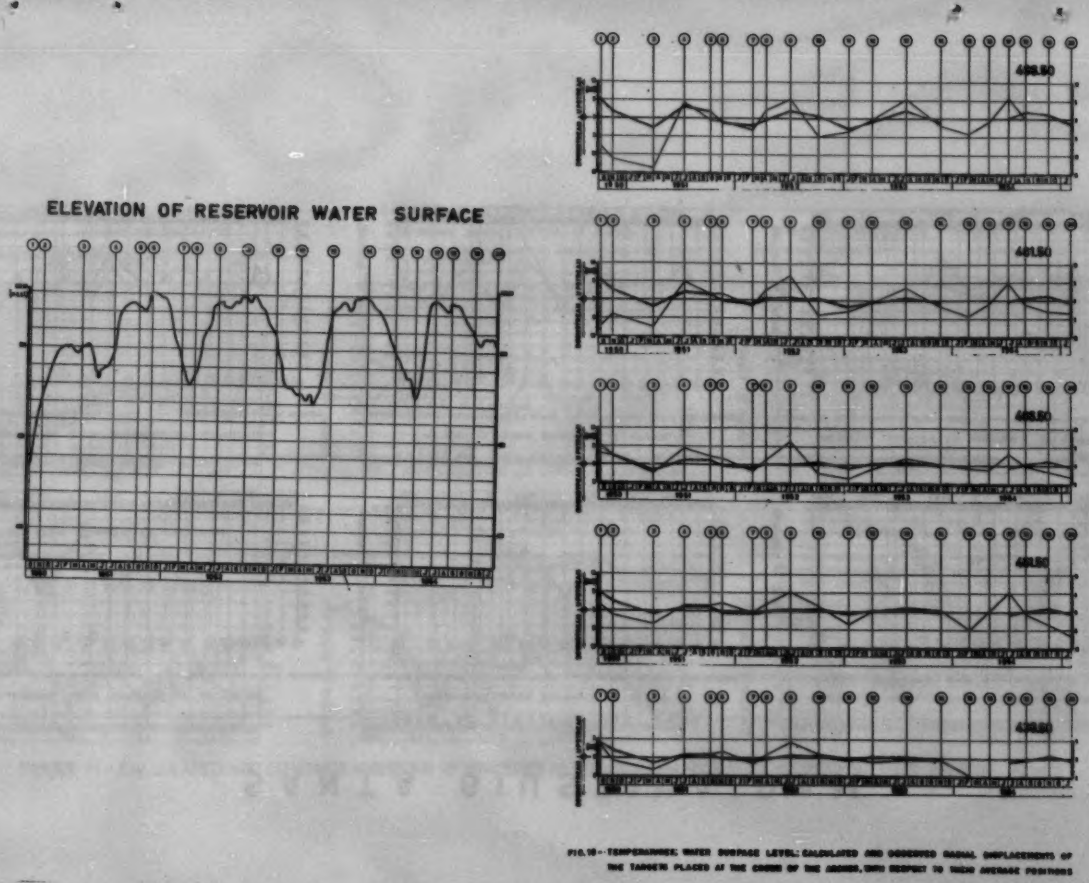
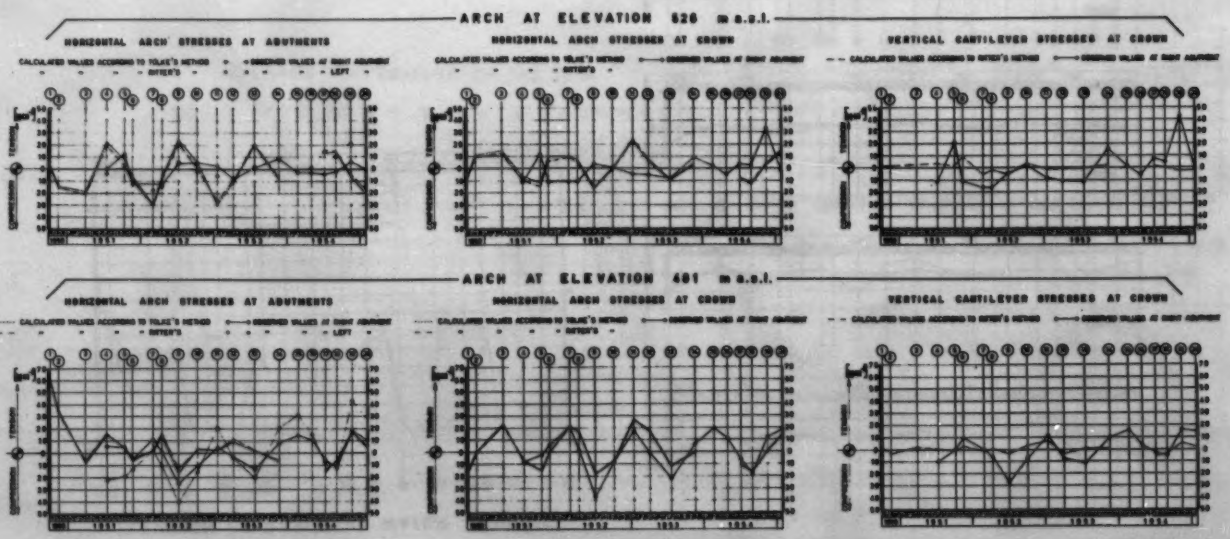


Fig. 10. Temperatures; water surface level; calculated and observed radial displacements of the targets placed at the crown of the arches, with respect to their average positions.

SANTA GIUSTINA DAM


ARCH AT ELEVATION 526 m a.s.l.
HORIZONTAL ARCH STRESSES AT ABUTMENTS
HORIZONTAL ARCH STRESSES AT CROWN
VERTICAL CANTILEVER STRESSES AT CROWN
ARCH AT ELEVATION 481 m a.s.l.
HORIZONTAL ARCH STRESSES AT ABUTMENTS
HORIZONTAL ARCH STRESSES AT CROWN
VERTICAL CANTILEVER STRESSES AT CROWN

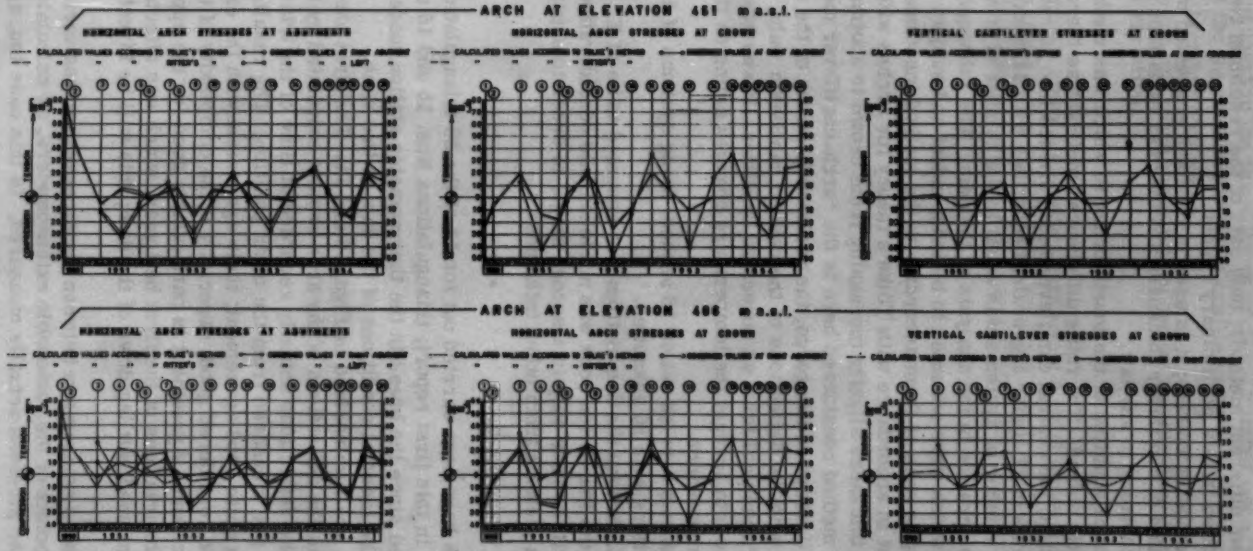

ARCH AT ELEVATION 451 m a.s.l.
HORIZONTAL ARCH STRESSES AT ABUTMENTS
HORIZONTAL ARCH STRESSES AT CROWN
VERTICAL CANTILEVER STRESSES AT CROWN
ARCH AT ELEVATION 406 m a.s.l.
HORIZONTAL ARCH STRESSES AT ABUTMENTS
HORIZONTAL ARCH STRESSES AT CROWN
VERTICAL CANTILEVER STRESSES AT CROWN

FIGURE 11 - CALCULATED AND OBSERVED VARIATIONS OF STRESSES AT DOWNSTREAM FACE, WITH RESPECT TO THEIR AVERAGE VALUES

In these calculations, the behaviour of the dam under water level and temperature variation was studied taking into account its tridimensional shape according to four different methods; only part of the results of two of these methods are stated in this preliminary report.

The first method is Tölke's; it is used in the form suggested by this author in the case of cantilever thickness and thermal variations depending on elevation according to any law.

The differential equation of the fourth order to which this method leads, cannot be exactly solved. The finite differences method was therefore used for its integration, the central cantilever being divided into 16 sections. In this way, systems of 21 linear algebraical equations with 21 unknowns were obtained. The number of systems was equal to the number of triangulations taken into consideration in the analysis of the dam. The subsequent analysis of the arches, which were assumed to be independent and subject to the actions pertaining to each of them according to the method mentioned above, was carried out in accordance with Tölke's theory for arches with a circular centerline and thickness slightly increasing from crown to abutments.

The second method considered here is the "arch-cantilever method." On account of the U-shape of the gorge, the radial deflections of the arches were made to agree with the deflections of the central cantilever only, according to this method's original form as conceived by Ritter. It was however generalized to take into account the temperature changes acting on the dam, besides the water pressure.

The analysis was carried out on 16 arches, using systems of 16 linear algebraical equations with 16 unknowns.

In these calculations the dam bottom was assumed to be at El. 382 m a.s.l.

Yielding of the foundation rock was not taken into consideration in either method; this will be done only in a second stage, in order to separate the influence of rock yielding from other influences.

* * *

Calculations have been carried out for 18 of the 20 triangulations which are mentioned in this first report; triangulations Nos. 15 and 16 could not in fact be analyzed since the break in the thermometer station made it impossible to record the thermal conditions of the dam at that time.

The results are summarized in Figs. 10 and 11. The first shows a comparison between the deflections of the arch crowns calculated by the two methods, and the deflections actually recorded at time of the triangulations; all deflections are measured from the mean position. The results of the two methods are in quite good agreement; their agreement with the experimental results must be considered good, especially if the complexity of the problem is taken into account. Agreement is particularly good for the upper arches, except in the observations of the first half year, which was affected by the previously mentioned settling down of the dam; this will be investigated further.

Fig. 11 shows a comparison between the stresses (stress variations) at the measurement points of the removable extensometers, as calculated by the two methods, and the stresses actually measured. In this case too, agreement between the two methods is very good. As was to be expected, the agreement

of the results of these methods with the experimental data is not very good, as far as the abutments are concerned; whereas it is fairly good for σ_a at the crown, at least in the three lower arches. Furthermore, the diagrams on the right of the figure prove that the cantilever is actually more heavily loaded than both methods show.

Stresses too are much more influenced by temperature changes than by water level variation.

* * *

As was stated above, this paper gives only a brief account of some of the results of a much more considerable analysis which is still being carried out; more definite conclusions are expected after this analysis is completed.

Paper 994 PO 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: THE RENO DI LEI DOUBLE-CURVATURE ARCH DAM

Claudio Marcello,[1] M. ASCE
(Proc. Paper 994)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 994 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

SYNOPSIS

This paper outlines the design by the Trial-Load Method of an unusual Italian arch dam. The centerlines of the horizontal arches are shaped to approximately coincide with the funicular curves of the arch loads. Design is not yet complete and will be verified by model studies.

The purpose of the Reno di Lei dam is to create in the Lei Valley (Retic Alps), at an elevation of over 1900 m a.s.l., a seasonal reservoir with a storage capacity of 200 million cubic metres, for regulation of the output of a system of 3 hydroelectric plants in series, with power station at Innerferrera, Andeer and Sils.

These plants belong to the Kraftwerke Hinterrhein A.G., a Swiss-Italian Company in which Soc. Edison of Milan also partakes.

The design of the dam was developed in 1954-1955. A small-scale model of the dam is now under construction to provide a mean for checking and completing design calculations and to suggest the possible adjustment of the structure design capable of improving its statical behaviour.

The main geometrical data of the designed arch dam are:

– maximum crest height above the deepest point of foundations	138 m
– thickness at crest	15 m*
– thickness at base	28 m
– length of crest arch	635 m
– chord of crest arch	520 m
– rise of crest arch	160 m
– volume of concrete	800,000 m^3

The crown section of the upstream face projects by 28 m at the crest level and by about 5 m at the base (Fig. 3).

The horizontal sections of the dam are arches of uniform thickness from crown to abutments. Their centerline coincides approximately with the funicular curve of the load supported by the arches, (see "Statical analysis"). The crest is at El. 1932, which is the maximum flood water level; a 2 meter high concrete wall will provide adequate protection against possible waves.

On the left bank, from a point about 13 m below the crest elevation, the dam will be placed against a concrete thrust block.

For construction purposes the dam body will be divided into blocks by temporary joints to minimize the tensile stresses due to shrinkage.

These joints will be about 15 m apart, and, after shrinkage, will be filled with concrete and grouted.

Possible pore pressure will be eliminated, or at least minimized, by means of a system of vertical drain pipes arranged behind the upstream face; drainage collectors and inspection galleries are also provided in the dam body (Fig. 4).

---

*This value of thickness was fixed by the Swiss military authorities, since the basin laying below the reservoir is in Swiss territory.

Statical Analysis

The stability analysis was made taking into consideration the reciprocal contribution to stability of the horizontal sections (arches) and of the vertical sections (cantilevers) (Fig. 6).

The dam was divided into 8 equally spaced horizontal arches of one unit height and into 15 vertical cantilevers. The dam is perfectly symmetrical with respect to the vertical plane at the crown of the arches and therefore it has been possible to limit the calculations to the cantilevers of only one half of the dam (8).

The arches are parabolic, of even thickness, and the equation of their centerline is of the type $y^2 = Wx$, where the parameter W is variable in function of the elevation.

The cantilevers are vertical dam sections of one unit width: their centerline is the intersection of the surface containing the centerlines of the arches with a vertical plane approximately normal to the dam faces. At each elevation, the thickness of the cantilever is equal to the thickness of the corresponding arch. Each cantilever has its base at an elevation which is the arithmetical average of the elevations of two consecutive arches.

The analysis has in general been carried out on the basis of the trial-load method, revised to fit parabolic arches and suitably simplified; in particular, deformations due to shear and twist actions have not been considered in the analysis of both the arches and the cantilevers, whereas the elastic movements of abutments and foundations have been considered, taking into account the tilting induced by the bending moment and shear, as well as the displacements induced by normal stress, shear, and bending moment.

At the foot of each cantilever the concrete was assumed not to be able to withstand tension as if the cantilever were cracked; consequently the active section at the base, in respect to the capacity of being deformed of the foundations, is only the area which will ultimately result in being subjected to compression, and was therefore determined by a series of approximations.

For the calculation of the factors of influence of the arches and cantilevers, the load assumptions were: unit forces at the intersection points, for cantilevers, and funicular loads with unitary mean value, for arches. Funicular loads are loads having a funicular curve coincident with the arch centerline.

The subdivision of the hydrostatic load between arches and cantilevers was obtained by bringing into agreement the horizontal cantilever deflections and the projection of the arch deflections onto the center plane of cantilevers, and then checking the results obtained against the assumptions made as regards both the reacting section at the foot of the cantilevers and the load distribution on the arches.

Only the deflections due to the radial components of the parts of hydrostatic load pertaining to the arches and to the cantilevers were considered.

A 5% allowance was accepted in the agreement of deflections in order to make the load surface more uniform since, owing to the simplifications assumed at the beginning, the perfect identity of deflections would have produced unacceptable irregularities on this surface. Anyway, a more accurate calculation will be carried out by means of an electronic computer, to obtain

the complete agreement of arch and cantilever deflections.

The stability analysis of the arches was then made by the theory of elasticity, taking into account the parts of hydrostatic load and the effects of the elastic yielding of abutments, and assuming a sinusoidal variation of the temperature between $\pm$ 10°C.

The analysis of the lower arches, these being the thicker ones, was carried out with the methods suitable for thick arches and taking into consideration only their part of hydrostatic load.

No allowance was made for shrinkage effects because provision of joints was made. These will be sealed only after complete concrete curing and under temperature conditions close to the average of the expected temperature fluctuations.

The analysis of the cantilevers was made taking into account the part of hydrostatic load, the dead weight of the cantilevers and the uplift pressure, assumed as linearly variable through the thickness.

The maximum stresses are recorded in the following tables. For the arches, the peak values were 74 Kg/cm^2 for compressive stress and 6 Kg/cm^2 for tensile stress in the assumption of simultaneous hydrostatic pressure and thermal effect at the maximum assumed values. With an empty reservoir the maximum value of thermal stress is very small ($\pm$ 2 Kg/cm^2).

For the cantilevers, the maximum values were 60 Kg/cm^2 for compressive stress and 18 Kg/cm^2 for tensile stress respectively.

Model

As mentioned above, a small-scale model of the designed dam is now being constructed. It will be made of a suitably processed concrete.

This model will undergo the statical tests necessary to determine the deflections, stresses and layout of isostatical lines of both full and empty reservoirs.

Ultimate strength tests will also be carried out as far as breaking point.

Waterproofing

The dam is to be built on a site consisting of paragneiss and mica-schist rock banks with strata parallel to the slope on the western bank and normal to the slope on the eastern bank.

The rock, as far as stability and watertightness are concerned, is generally sound.

A layout of waterproofing and consolidation groutings is being studied, based on a careful geological survey, and on the outcomes of a set of core drillings purposefully carried out.

The design of the dam was entrusted to Soc. Edison and performed by this Company's Hydroelectric Plant Construction Department which is directed by the writer.

It is in the intention of the writer to go farther into the object of this paper as soon as the design of the dam is finally completed.

RENO DI LEI
Parabolic arch dam -

TABLE OF STRESSES ON THE ARCHES

Kg/cm^2

a) Elastic arches

Arch No.	stresses	Pressure on the arch		Pressure on the arch and temperature changes				Temperature changes with empty reservoir	
		Crown	Abutment	Crown		Abutment		Crown	Abutment
				Summer	Winter	Summer	Winter		
8	e	33.10	54.72 (17.73)	28.78	37.42	51.63 (14.19)	57.81 (21.27)	$\mp$ 0.46	$\pm$ 0.72
	i	48.77	27.07 (64.00)	53.13	44.41	30.19 (67 57)	23.95 (60.43)	$\pm$ 0.47	$\mp$ 0.69
7	e	39.65	24.52	35.36	43.94	21.57	27.37	$\mp$ 0.55	$\pm$ 0.88
	i	42.13	57.61	46.48	37.78	60.50	54.72	$\pm$ 0.61	$\mp$ 0.92
6	e	45.41	16.91	41.24	49.58	14.42	19.10	$\mp$ 0.62	$\pm$ 1.06
	i	35.54	64.57	39.80	31.28	67.12	62.02	$\pm$ 0.71	$\mp$ 1.00
5	e	46.69	8.36	42.57	50.81	6.30	10.42	$\mp$ 0.73	$\pm$ 1.30
	i	28.37	67.26	32.63	24.11	69.42	65.10	$\pm$ 0.88	$\mp$ 1.20
4	e	50.73	-4.57	46.62	54.84	-5.93	-3.21	$\mp$ 0.89	$\pm$ 1.68
	i	15.93	72.70	20.36	11.50	74.29	71.11	$\pm$ 1.18	$\mp$ 1.47

Note: the bracketed figures are referred to the section where the curve of pressures shows the maximum of excentricity.

b) Rigid arches

Arch No.	Pressure		Stresses (Kg/cm^2)				
	Hydrostatic t/m^2	on the arch t/m^2	After Mariotte	After Lamé		After Résal	
			for hydrostatic pressure	for pressure		for pressure on the arch	
				hydrostatic	on the arch	Crown	Abutment
3	94.875	51.819	71.10	76.18	41.29	51.52	68.69
2	112.125	80.477	52.50	58.78	42.19	47.83	63.77
1	129.375	196.375	30.18	38.42	58.32	20.10	26.80

RENO DI LEI
Parabolic arch dam -

TABLE OF STRESSES ON THE CANTILEVERS

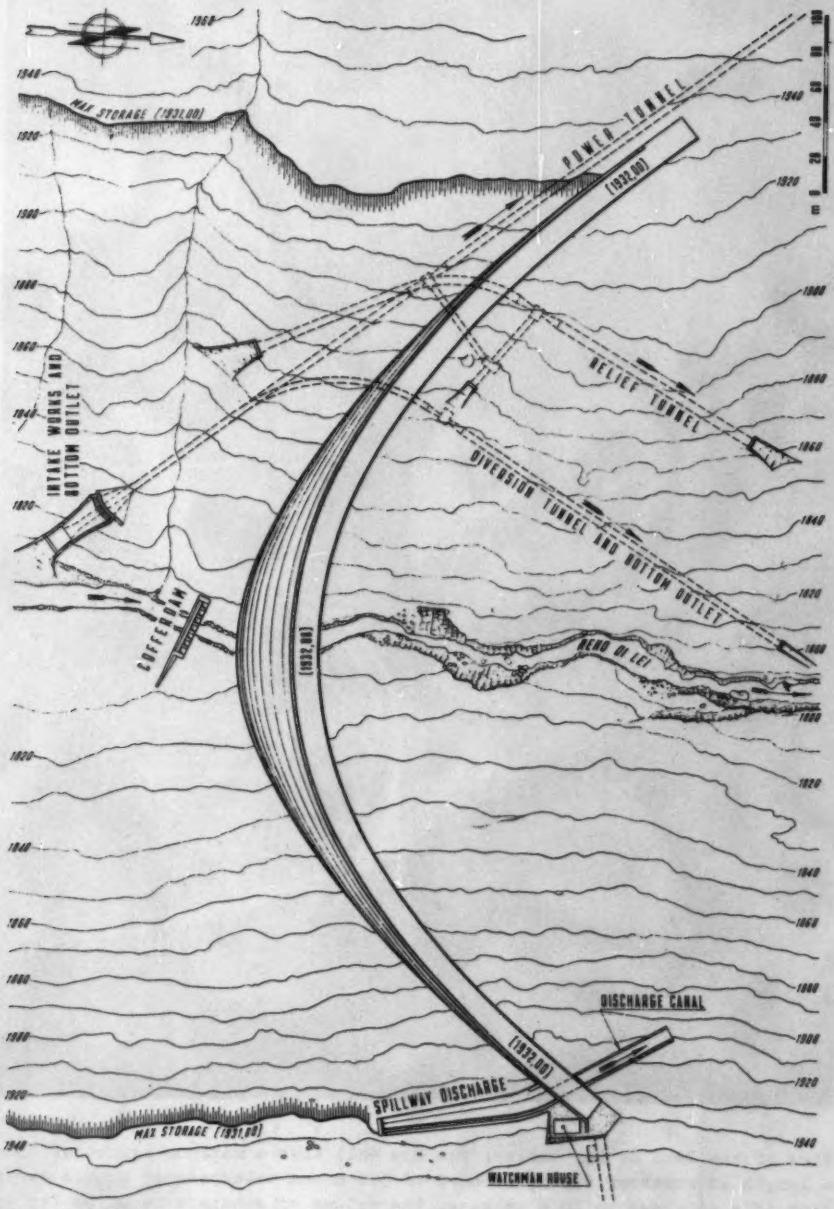
Kg/cm^2

Arches	Cantilevers															
	1		2		3		4		5		6		7		8	
	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream
8	5.03	2.22	5.47	2.15	5.58	1.55	5.43	1.65	3.92	2.95	1.74	4.48	0.32	6.12	-	7.20
7	I2.24	I.72	I4.4I	-0.6I	I4.26	-I.26	I2.43	0.46	9.33	2.82	-I.5I	I3.93	-	2I.39		
6	25.29	-3.26	22.94	-4.60	20.75	-4.06	I6.06	-I.07	4.62	I2.07	-	40.03				
5	I9.50	2.08	26.40	-6.30	I8.52	-0.49	2.56	I6.7I	-	37.29						
4	I4.73	5.88	I6.47	4.76	2.2I	I9.54	-	60.98								
3	-5.49	29.76	-0.72	24.00	-	56.58										
2	-I8.24	43.84	-	45.9I												
I	-	50.98														

FIG. 1

View of the Reno di Lei Valley. The dam will have a maximum height of 138 m, a length at crest of 635 m, a chord of 520 m and thickness of arches varying from 15 m at crest to 28 m at base. The volume of concrete required for the construction of the dam is 800,000 m^3.

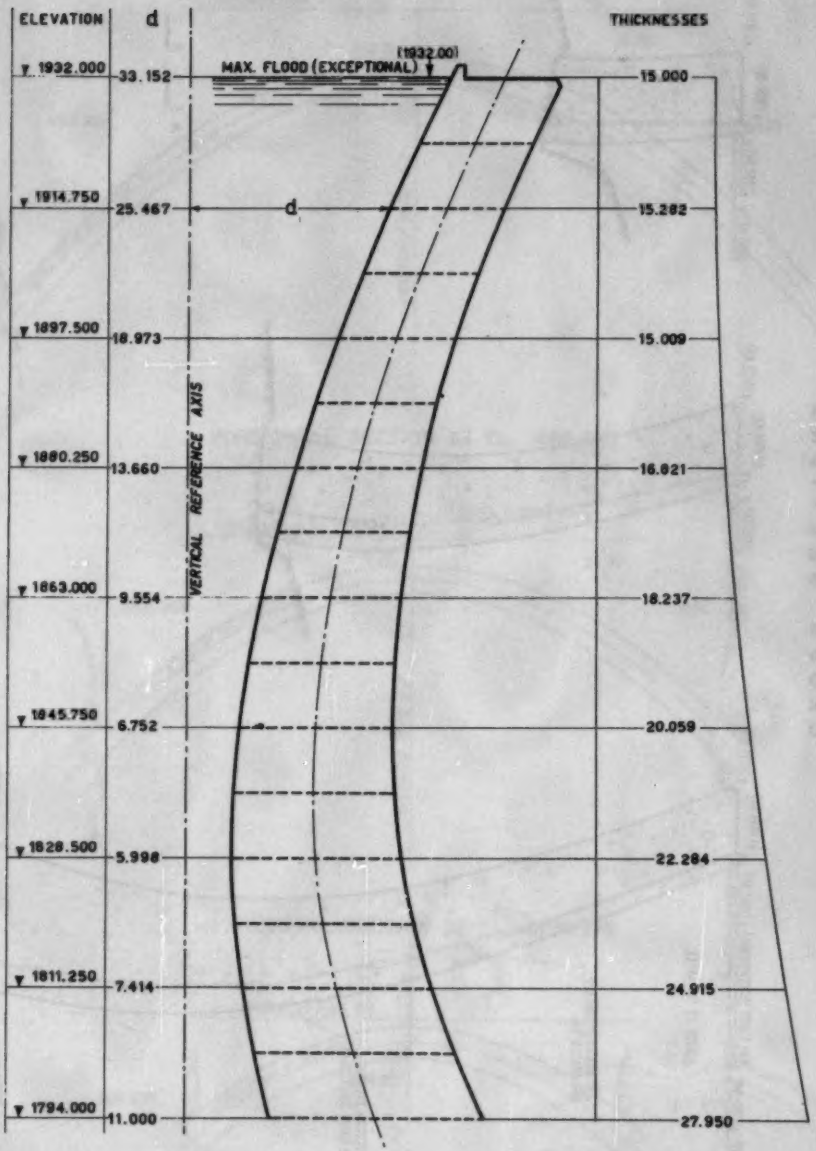
GENERAL LAYOUT FIG. 2



RENO DI LEI DAM

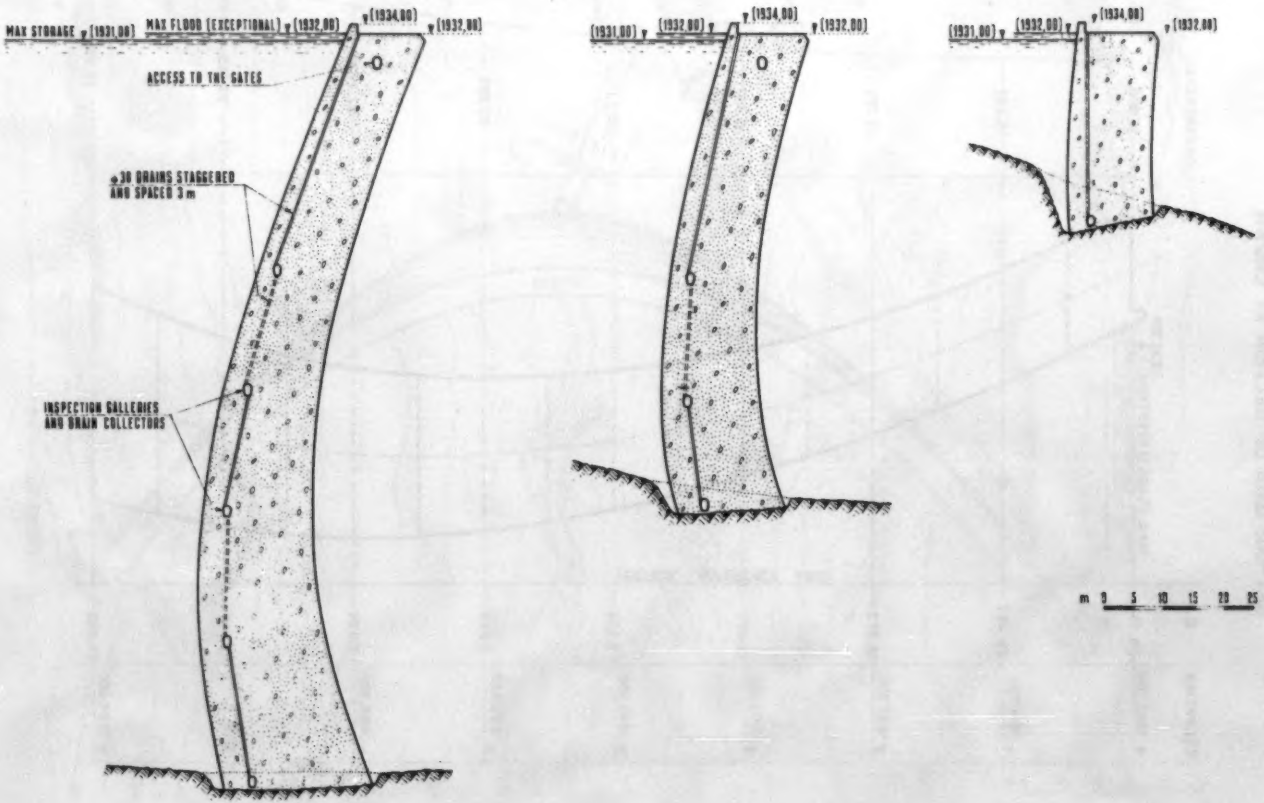
FIG. 3

SET-OUT DATA OF SECTION AT CROWN



RENO DI LEI DAM

CROSS SECTIONS



994-10

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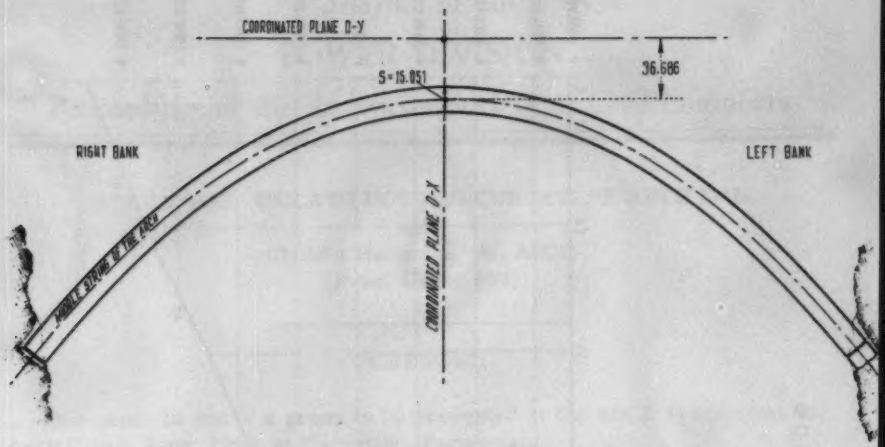
June, 1956

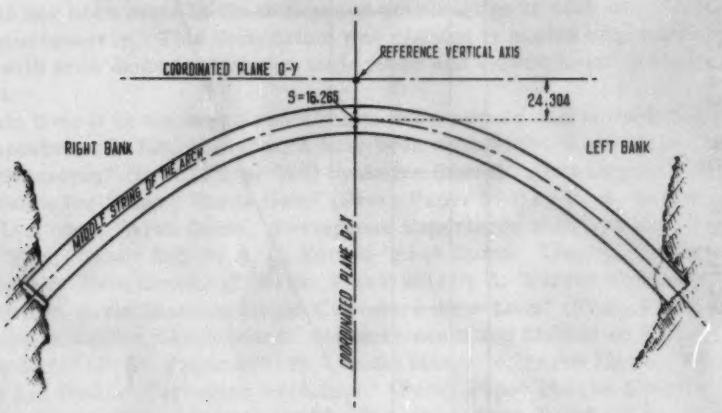
FIG. 4

FIG. 5

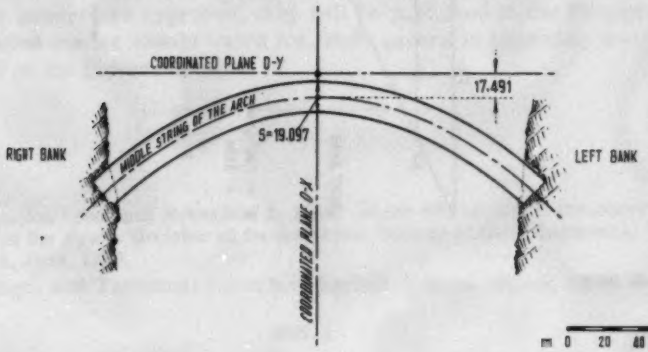
HORIZONTAL SECTION AT EL. 1923.375



HORIZONTAL SECTION AT EL. 1888.875



HORIZONTAL SECTION AT EL. 1854.375



SCHEME OF CALCULATION

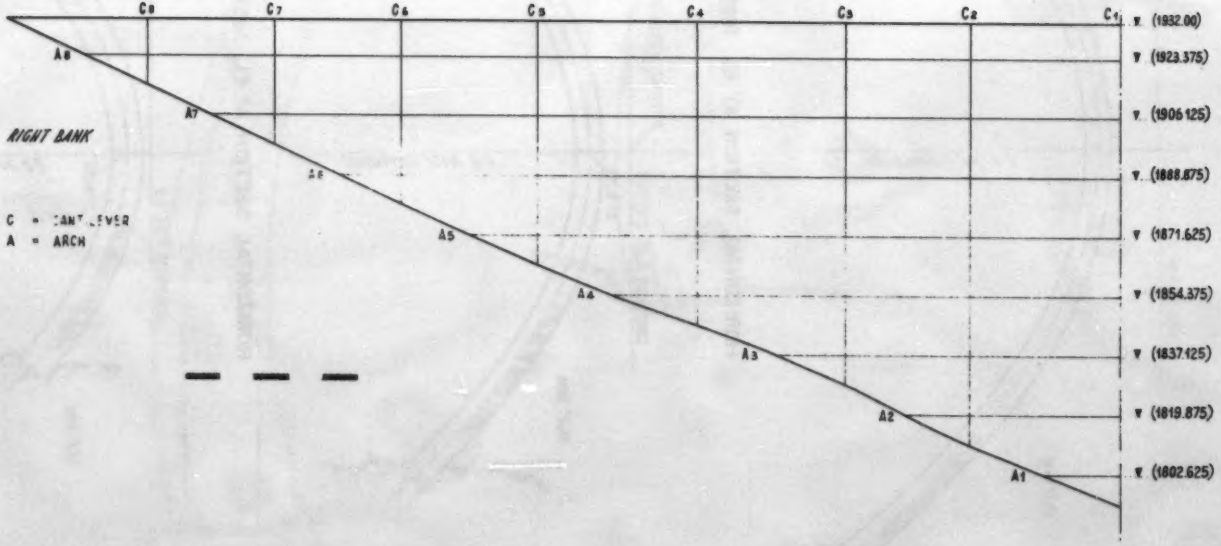


FIG. 6

Paper 995 PO 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: ISOLATO DOUBLE-CURVATURE ARCH DAM

Claudio Marcello,[1] M. ASCE
(Proc. Paper 995)

FOREWORD

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1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

SYNOPSIS

This paper outlines the design and construction of an Italian arch dam 152 meters high and located in an extremely narrow gorge. Briefly discussed are the structural analysis of the dam—confirmed by model, construction plant, and the extensive foundation consolidation and cut-off grouting.

General

The Isolato double-curvature arch dam was constructed in the summers of the years 1952 and 1953 in order to provide a reservoir for the monthly regulation of the inflow of the basin harnessed by the Liro hydroelectric plant No. 3 in the Spluga Valley (Lepontine Alps) owned by Soc. Edison.

The dam was designed in 1950 and the corresponding model was tested from the statical point of view in the Laboratory of the Politechnic School of Milan.

The dam, shown on Fig. No. 2, has a maximum height of 132 ft, 40.50 m above foundation level and a thickness of 3.93 to 2.06 m at the crown. The upstream face has a single center, with radii variable from 18.88 to 37.46 m and center angles variable from 66.264 to 148.165 centesimal degrees, while the downstream face has three centers below El. 1243 and a single center above this elevation.

Contraction joints divide the dam into three blocks below El. 1233 and five blocks above this elevation (see Fig. No. 3). These joints were grouted in the winter following the concrete pouring and were made watertight by means of U-shaped 2 mm copper sheets embedded in the concrete.

Figs. No. 1 and 4 show the dam as seen from downstream and from upstream respectively.

Statical Analysis

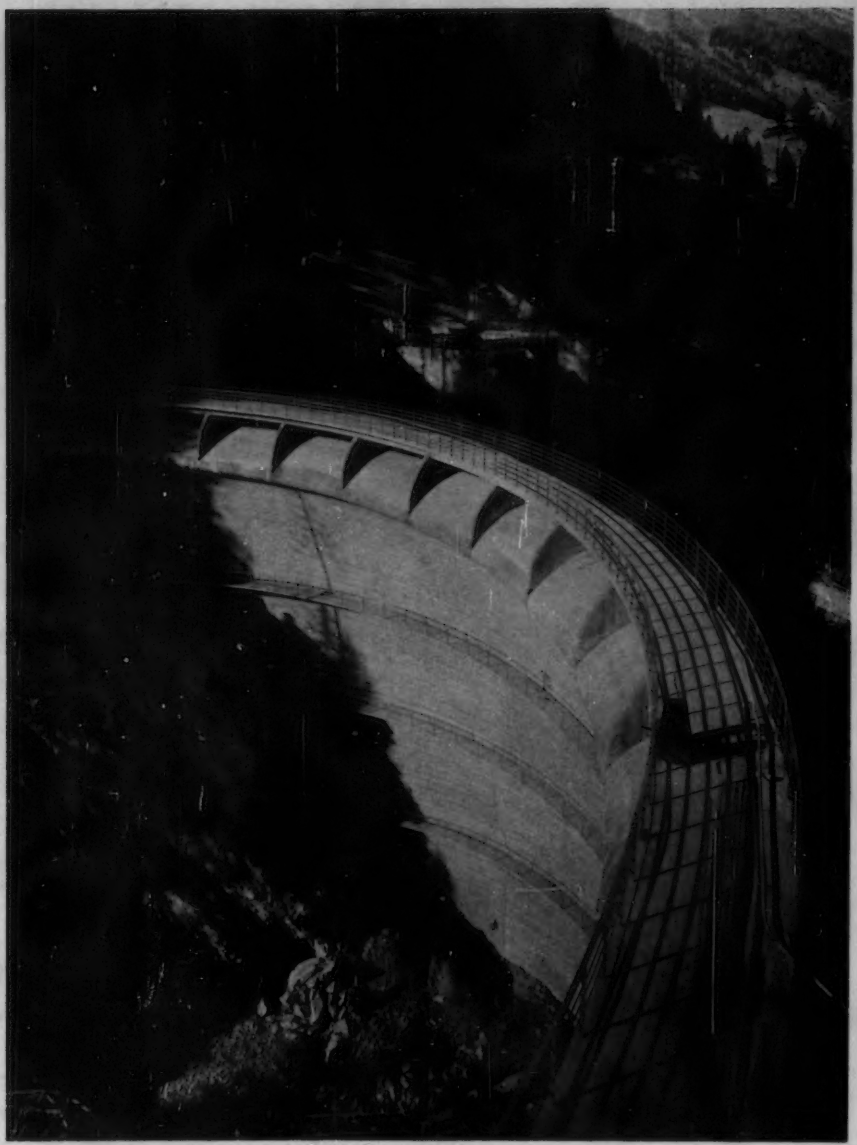
The statical analysis was carried out on individual arches keyed into the abutments, normal to the centerline of the crown cross section, subject to hydrostatic pressure, dead weight, temperature changes and shrinkage of concrete.

This analysis, which makes no allowance for any vertical solidarity of the dam, is justified by the comparatively small size of the gorge and consequently also of the thicknesses of the structure.

Fig. No. 5, of the cross section of the dam, shows the arches which have been checked.

The following factors have been considered:

– specific gravity of water	1.00 ton/cu.m
– specific gravity of concrete	2.50 ton/cu.m
– annual variation in temperature	$\pm$ 10° C
– daily variation in temperature	$\pm$ 6° C
– concrete shrinkage during setting made equal to a uniform decrease in temperature of	3° C



View from downstream. The dam has a maximum height of 40.50 m and a thickness at crown variable from 3.93 m to 2.06 m. In the background, the spillway with automatic sector gate.

GENERAL LAYOUT

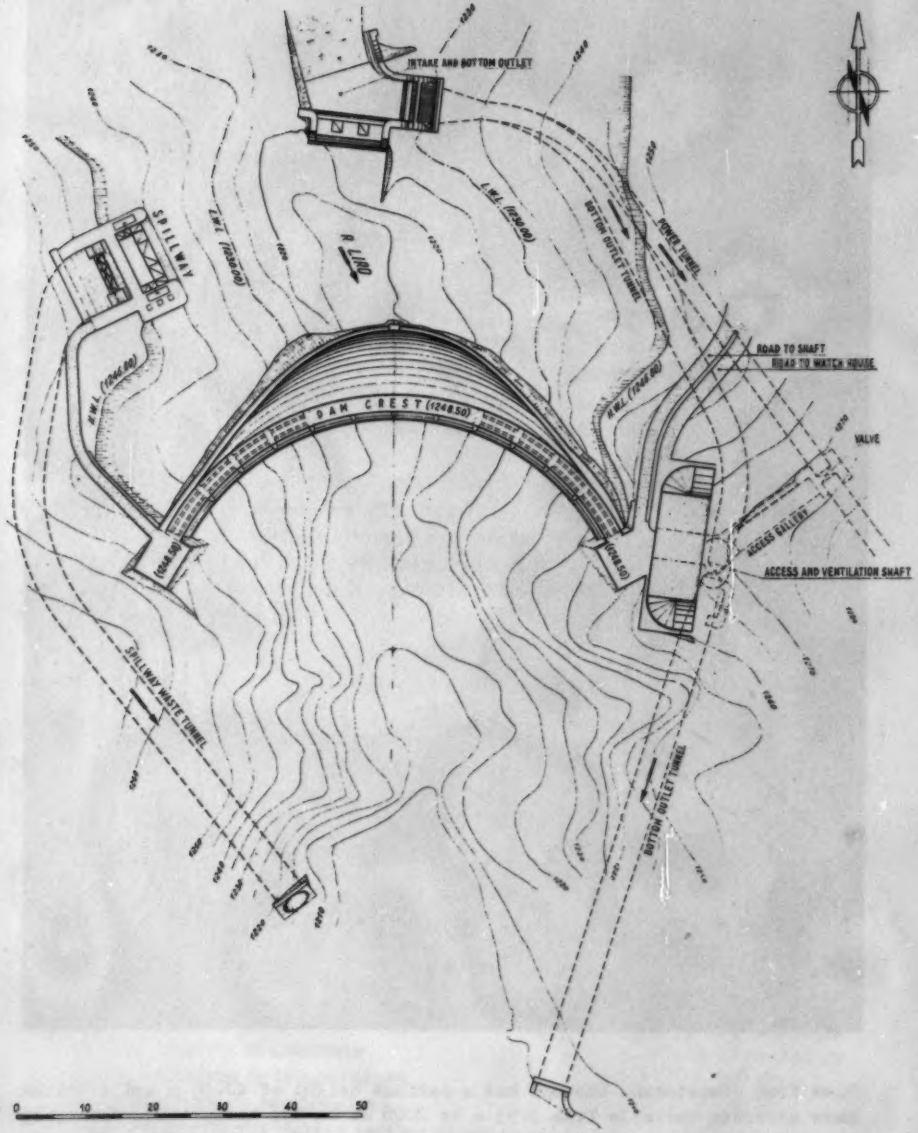
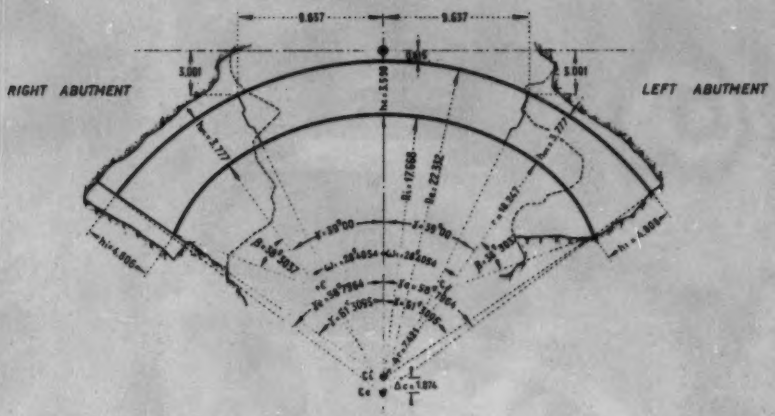


FIG. 3

HORIZONTAL SECTION AT EL. 1218.00



HORIZONTAL SECTION AT EL. 1233.00

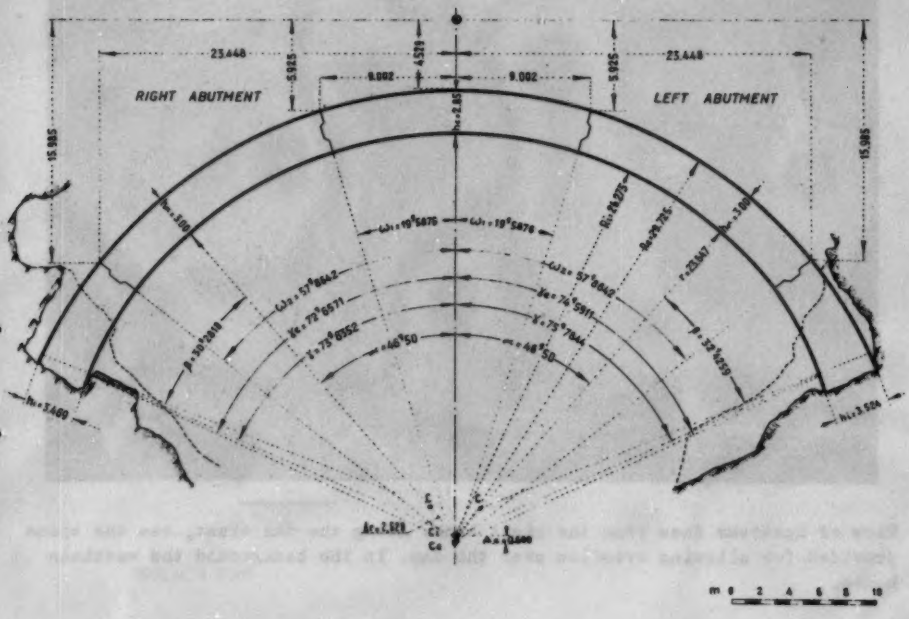
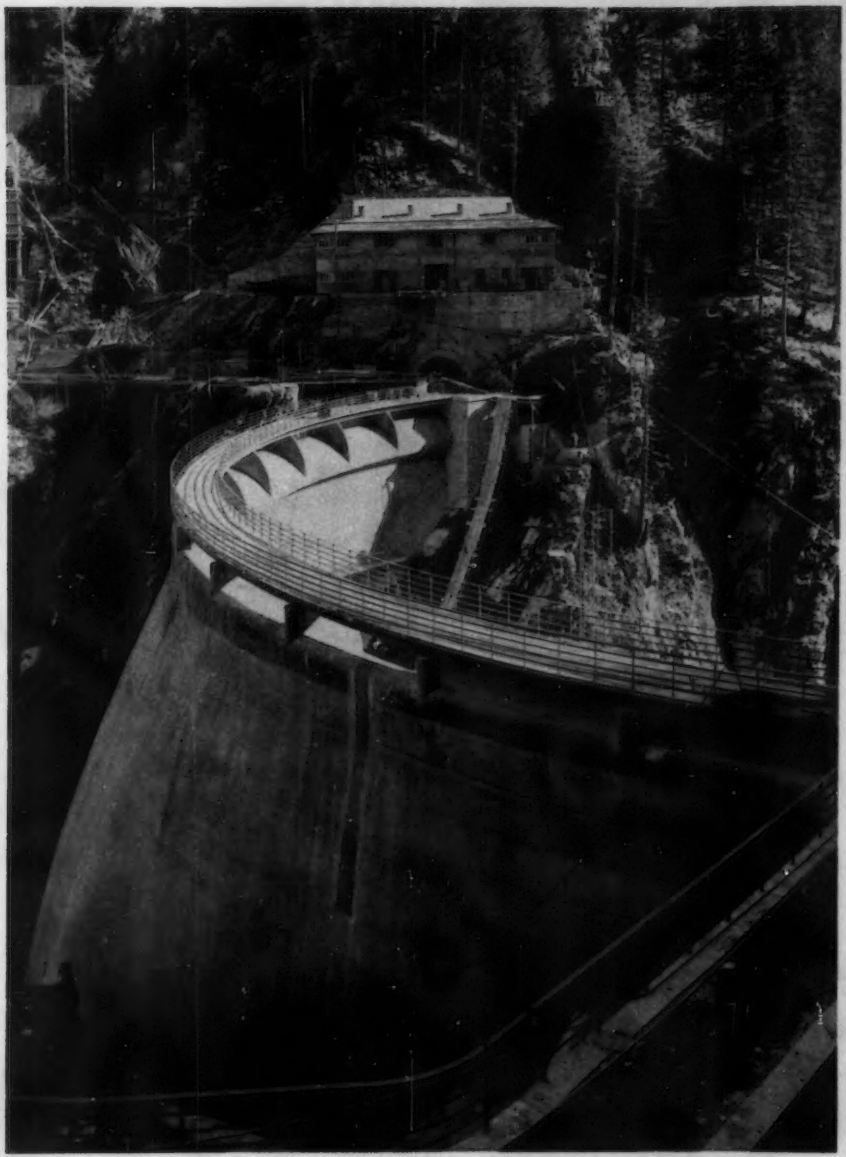


Fig. 4

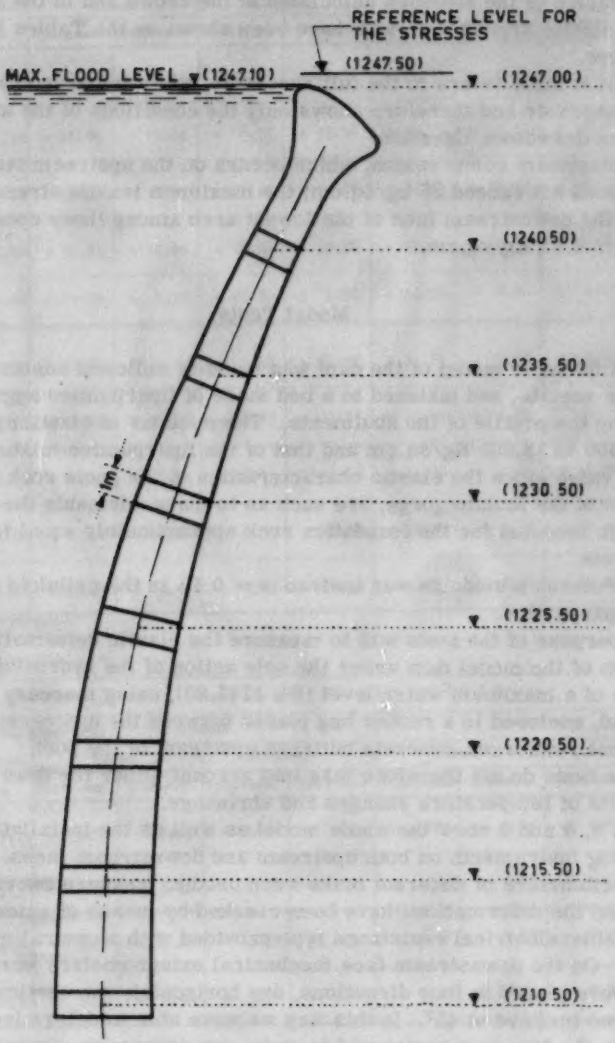


View of upstream face from the right bank. Along the dam crest, see the spans provided for allowing overflow over the dam. In the background the watchman house.

CROSS SECTION WITH THE TRACED PLANES
NORMAL TO THE MEDIUM FIBER
DETERMINING THE CHECK RINGS

SCALE 1:200

Fig. 5



ISOLATO DAM

For the stability analysis of the 2 deepest horizontal arches (at El.1210.50 and 1215.50) which were assumed as stiff arches, we have followed Résal's hypothesis.

As far as the upper arches are concerned, which have been considered as elastic independent arches, we have allowed at first approximately for a stiffening effect produced by the crest arch, ensuring the coincidence between the displacements at the crown of the crest arch and of the two underlying horizontal arches of equal thickness.

The values of the stresses calculated at the crown and in the abutments of the five elastic arches analyzed, have been shown on the Tables No. 1 and 2 here below.

The first table refers to the full reservoir, while the second refers to the empty reservoir and therefore shows only the conditions of the arches above maximum drawdown elevation.

The maximum compression, which occurs on the upstream face at the crown, does not exceed 35 kg/sq.cm; the maximum tensile stress, which occurs on the downstream face of the lowest arch among those considered elastic, is 6.62 Kg/sq.cm.

Model Tests

The 1:50 scale model of the dam was made of celluloid sheets jointed with cellulose acetate, and fastened to a bed made of liparpumice aggregate reproducing the profile of the abutments. The modulus of elasticity of celluloid was 17,500 to 18,000 Kg/sq.cm and that of the liparpumice mixture was of a similar value since the elastic characteristics of the gneis rock which forms the walls of the Isolato gorge, are such as to make advisable the adoption of an elastic modulus for the foundation rock approximately equal to the modulus of concrete.

The Poisson's modulus was instead $\nu = 0.40$ in the celluloid and $\nu = 0.15$ in the liparpumice.

The purpose of the tests was to measure the elastic deformations in the two faces of the model dam under the sole action of the hydrostatic load in the case of a maximum water level (El. 1246.80), using mercury as the loading liquid, enclosed in a rubber bag placed between the upstream face and a convenient reinforced-concrete buttress upstream of the pool.

These tests do not therefore take into account either the dead weight or the effects of temperature changes and shrinkage.

Figs. 6, 8 and 9 show the whole model as well as the installation of the measuring instruments on both upstream and downstream faces.

Deflectometers of different make were used to measure the rises in the model and the deformations have been checked by means of extensometers of the variable-electrical resistance type provided with a central measuring system. On the downstream face mechanical extensometers were used. These were placed in four directions, one horizontal, one vertical and the others two inclined at 45°. In this way we were able to determine the stress tensor on the face in question and to make any necessary corrections in the readings.

TOTAL STRESSES ON THE INCLINED ELASTIC ARCHES WITH VARIABLE THICKNESSES AND VARIOUS TEMPERATURE DISTRIBUTIONS

TABLE No 1

Stresses in Kg/sq.cm. Sign + means compression; Sign - means traction

	El. of arch center-line	FULL RESERVOIR							
		SUMMER				WINTER			
		Crown		Abutment		Crown		Abutment	
		Upstream	Downstream	Upstream	Downstream	Upstream	Downstream	Upstream	Downstream
without shrinkage	1220.50	+ 18.76	+ 13.34	+ 6.45	+ 18.66	+ 31.57	- 1.35	+ 4.69	+ 19.30
	1225.50	+ 15.55	+ 15.34	+ 8.15	+ 15.56	+ 32.15	- 3.00	+ 7.11	+ 15.98
	1230.50	+ 18.57	+ 12.54	+ 9.46	+ 13.80	+ 32.87	- 2.65	+ 10.74	+ 12.31
	1235.50	+ 17.71	+ 14.21	+ 10.68	+ 10.08	+ 32.64	- 1.34	+ 12.48	+ 8.13
	1240.50	+ 21.19	+ 7.32	+ 10.22	+ 9.98	+ 33.58	- 5.24	+ 13.65	+ 6.51
with shrinkage (-3°C)	1220.50	+ 20.90	+ 10.12	+ 2.59	+ 22.17	+ 33.71	- 4.57	+ 0.82	+ 23.31
	1225.50	+ 17.64	+ 12.46	+ 4.71	+ 18.71	+ 34.14	- 5.88	+ 3.67	+ 19.13
	1230.50	+ 20.18	+ 10.31	+ 6.61	+ 16.50	+ 34.48	- 4.88	+ 7.89	+ 15.00
	1235.50	+ 19.14	+ 12.38	+ 8.59	+ 12.07	+ 34.07	- 3.17	+ 10.40	+ 10.12
	1240.40	+ 22.28	+ 5.93	+ 8.22	+ 11.91	+ 34.67	- 6.62	+ 11.65	+ 8.44

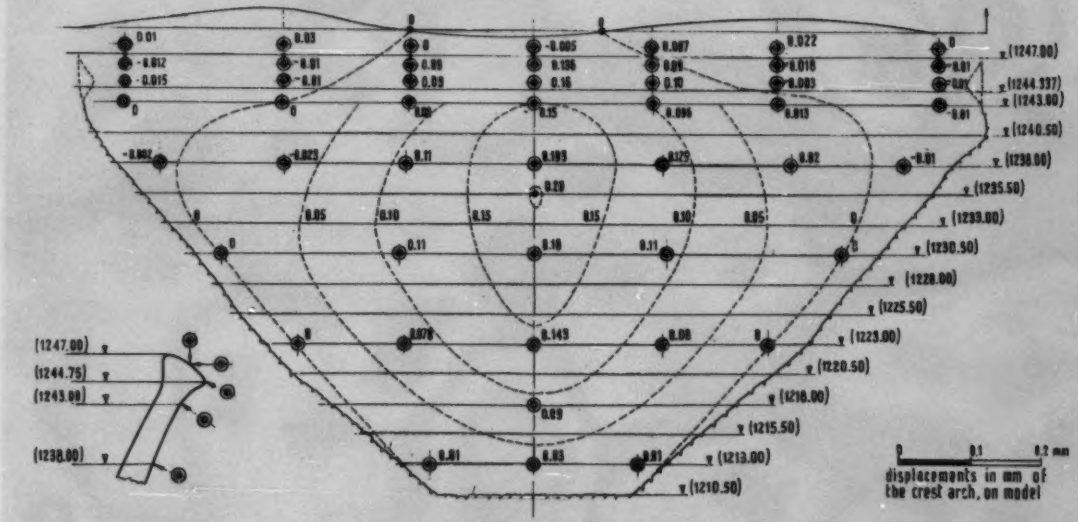
TABLE No 2

	El. of arch center-line	EMPTY RESERVOIR							
		SUMMER				WINTER			
		Crown		Abutment		Crown		Abutment	
		Upstream	Downstr.	Upstream	Downstr.	Upstream	Downstr.	Upstream	Downstr.
without shrinkage	1235.50	- 4.70	+ 6.03	+ 6.87	- 6.55	+ 4.70	- 6.03	- 6.87	+ 6.55
	1240.50	- 4.14	+ 5.28	+ 6.78	- 6.53	+ 4.14	- 5.28	- 6.78	+ 6.53
with shrinkage (-3°C)	1235.50	- 3.27	+ 4.19	+ 4.78	- 4.56	+ 6.12	- 7.86	- 8.95	+ 8.54
	1240.50	- 3.05	+ 3.90	+ 4.78	- 4.60	+ 5.23	- 6.67	- 8.78	+ 8.45

View of the upstream side of model constructed on the scale 1:50 with plates of celluloid jointed by means of aceto-cellulose and arranged on a proper bed reproducing the development of the abutments.

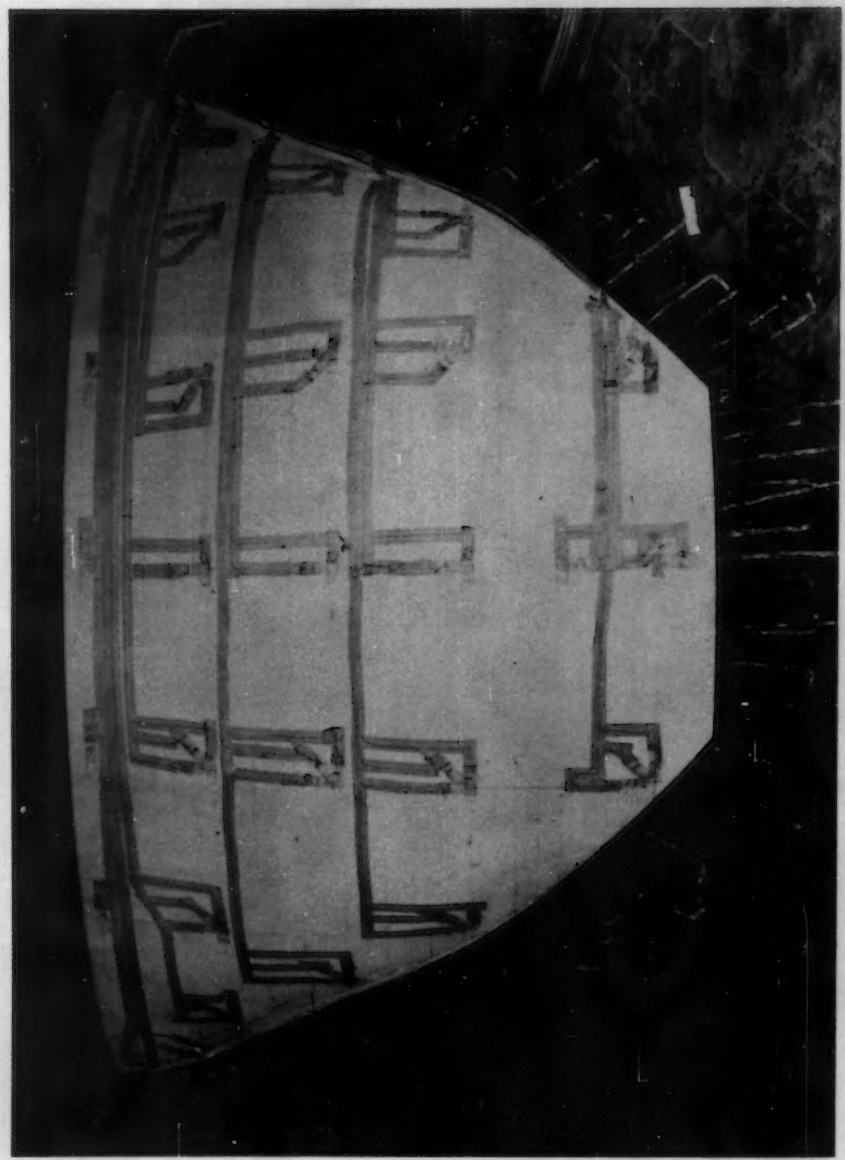
LENGTH OF DOWNSTREAM FACING

CURVES OF RADIAL DEFLECTION



DIFFERENCE OF LEVEL 0.05 mm ON MODEL = 0.05 cm ON PROTOTYPE WITH Ec = 320.000 kg/sq.cm

FIG. 7



View of the testing equipment installed on the upstream face of model.

View of the downstream face of model with the testing equipment arranged for measuring the deformations. See on the facing the extensometers and, upstream, the rubber bag containing the mercury used for applying the load on the structure.

The results obtained by this test are the following.

Radial deflections of the downstream face, after having drawn the necessary contour lines, show the essential symmetry of the structure, which acts like a double-curvature plate. The upper portion of the two shoulders shows the characteristics inversion of rises.

The maximum absolute value was found to be in the central part at El. 1235.50, i.e., at about 2/3 of the maximum height, and turned out to be approximately as small as 2 mm.

The horizontal stresses are compression stresses all over the upstream face and on the whole they are very small. The maximum stress is 42.7 Kg/sq.cm and was also found to be in the middle area as stated above.

By comparing this value with that of 25 Kg/sq.cm resulting from the calculation based on the assumption of independent arches, it may be seen how the double-curvature plate effect has raised the value of stresses in this area, consequently decreasing it in the peripheral zones.

Vertical stresses are always small; maximum compression (15.30 Kg/sq. cm) is found in the same area as that in which the maximum values in the arches have been recorded.

On the other hand tensile stresses occur at the bottom of the cantilever sections owing to the foundation being keyed. These tensile stresses however are not dangerous insofar as they are to be added to the compression stresses due to dead weight which have been estimated as being greater.

On the downstream face the horizontal stresses are on the whole compressive and are very weak, reaching the peak value of 20 Kg/sq.cm at the right abutment of the arch at El. 1223; tensile stresses occur only in the downstream middle area with a local peak of 5.3 Kg/sq.cm. Vertical stresses are weak and tensile in character all over a wide strip between El. 1230 and 1240 reaching El. 1215 in its lower central section; the absolute maximum of 5.1 Kg/sq.cm is found at El. 1223 while compressive stresses are present at the foundation of the cantilevers with a peak value around 12.50 Kg/sq.cm in the higher middle sections.

The stresses found on the centilevers have confirmed that these act as beams which are partially keyed at their foot and which elastically rest upon the upper arches.

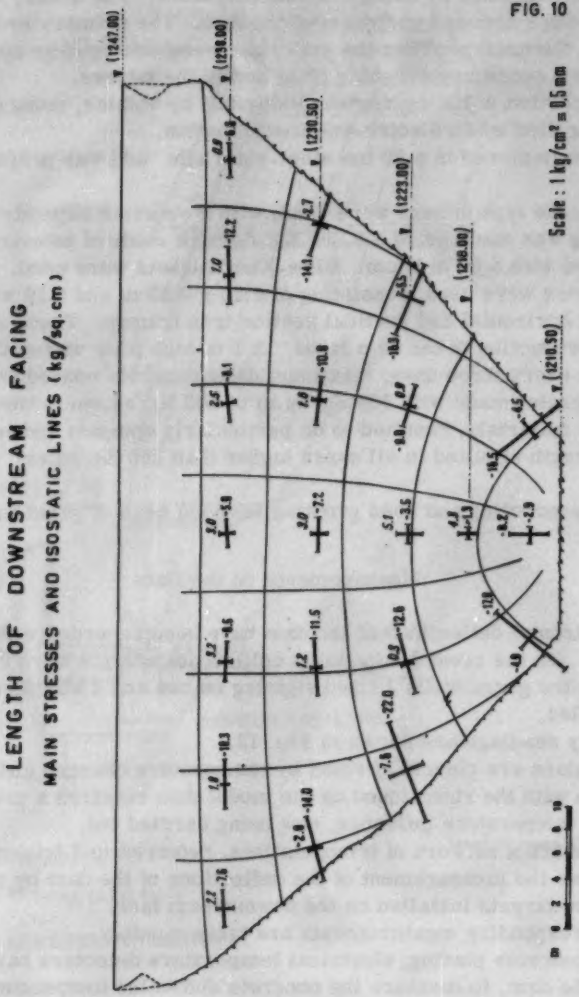
Fig. 10 shows the direction and intensity of the main stresses on the downstream face. The maximum main compression stress turns out to be 22 Kg/sq.cm and the maximum tensile stress 5.1 Kg/sq.cm, at the same points as previously indicated.

The isostatic lines are very indicative because once again they confirm both the characteristic spacial behaviour of the double-curvature structure and the symmetry of the stress interplay.

Construction

The construction of the dam required particular care. As regards excavations, we had to be sure to extend them only to a sound rock level, and this in the presence of a remarkably schistous formation. Excavations had to be carried out with a minimum amount of explosives and mostly by cutting the

FIG. 10


LENGTH OF DOWNSTREAM FACING
MAIN STRESSES AND ISOSTATIC LINES (kg/sq.cm)
Scale : 1 kg/cm² = 0.5 mm
(1247.00)
(1230.00)
(1230.50)
(1223.00)
(1216.00)
(1210.50)

rock with wedges and jackhammers, after having hammer-drilled a previous set of holes close to one another; these were not loaded.

As regards concrete pouring, care had to be taken owing to the overhanging surfaces and the double-curvature nature of the structure, which had to be made with great precision in respect to the very small thicknesses.

Concrete was made by using crushed limestone. The quarry was situated 350 m above the dam and upstream of the dam. The primary crushers were installed in the quarry. Then the materials were conveyed by continuous cableway to a secondary crushing plant and to the mixers.

The proportion of the aggregates was made by volume, using dosing machines controlled by an electro-pneumatic station.

Cement was stored in a 10 ton steel-plate silo and was proportioned by weight.

Double-cone type mixers were used, with a concrete capacity of 750 lt. The pouring was made using a 3,000 Kg. derrick made of tubular framework and provided with a 50 m boom. Blow-Knox buckets were used.

Steel forms were used, consisting of 0.50 x 0.50 m and 0.50 x 1.00 m panels, and horizontal and vertical section iron frames. These allowed a perfect reproduction of the dam faces. A 1 m high pour was made on the same block every three days; maximum daily concrete poured was 150 cu.m.

The concrete, made with 300 Kg/cu.m of 500 Kg/sq.cm cement and very rich in fine materials, resulted to be particularly compact and impervious and its strength resulted in all cases higher than 250 Kg/sq.cm after 28 days.

The waterproofing and bond grouting have not been of great importance.

Measurements on the Dam

The horizontal deflections of the dam have been recorded with a Fennel collimator. On the crest 2 adjustable collimation targets were embedded, whereas on the gorge walls 2 fixed sighting marks and 2 collimator stations were installed.

The daily readings are shown in Fig. 12.

Their values are clearly affected by temperature changes and therefore a comparison with the rises found on the model dam requires a preliminary analysis of temperature influence, now being carried out.

Furthermore a network of triangulations, referred to 7 triangulation bases, allows the measurement of the deflections of the dam by means of 18 triangulation targets installed on the downstream face.

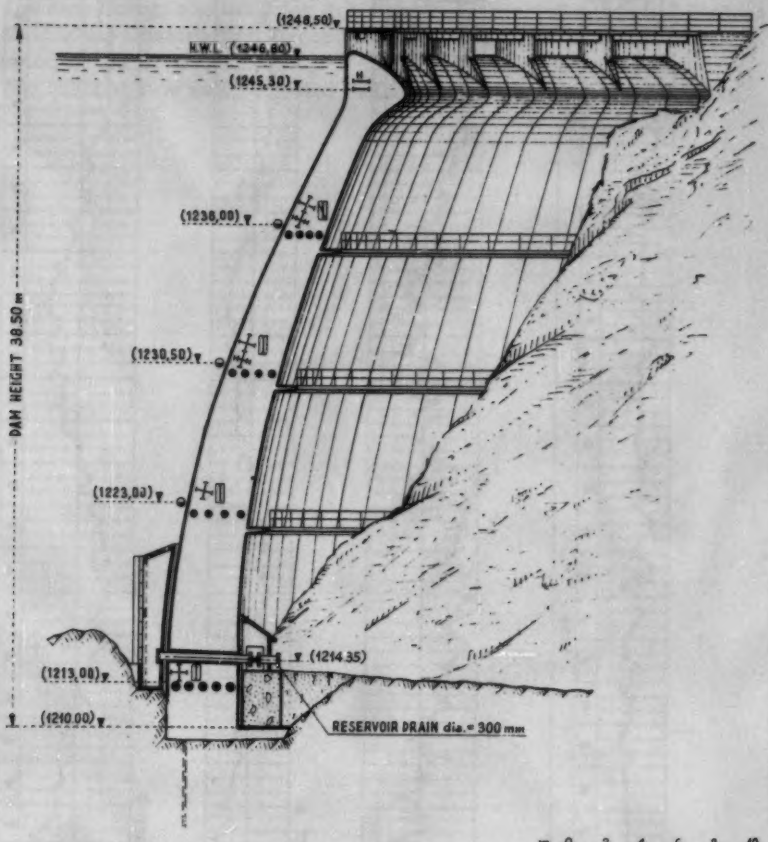
The corresponding measurements are taken monthly.

During concrete placing, electrical temperature detectors have been embedded in the dam, to measure the concrete and water temperatures. Also electroacustic extensometer ranged in vertical and horizontal directions as well as at inclinations of 45^{0}, for a complete determination of the stress tensor work.

Electroacustic extensometers isolated from the structure record the deformations due to temperature and humidity.

Fig. 11 shows the instrument embedded in the dam along the crown section.

POSITION OF MEASURING INSTRUMENTS FIG. 11



CHECK MEASUREMENTS ON THE BEHAVIOUR OF THE DAM

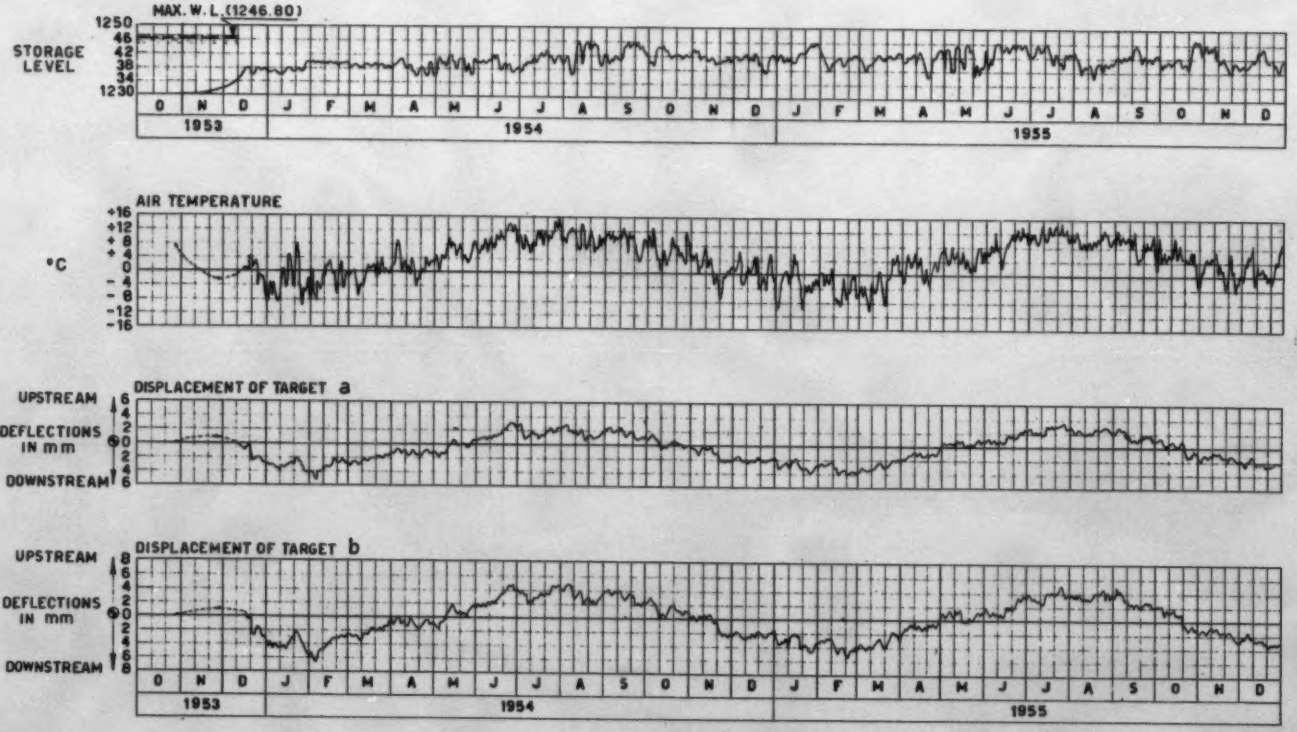


FIG. 12

A set of marks have been installed on the downstream face in order to measure the deformations along 4 directions at 45°, by means of a removable extensometer.

The dam design and the work direction were carried out by the Hydroelectric Plant Construction Department of Soc. Edison of Milan, which is directed by the writer.

The construction was entrusted to the contractor S.A.L.C.I. of Milan.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: RIO FREDDO DAM WITH GRAVITY ABUTMENTS AND CUT-OFFS

Claudio Marcello,[1] M. ASCE

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April, 1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, nine papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Single-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; and "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 996 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

SYNOPSIS

This paper outlines the design and construction of an Italian double-curvature arch dam with gravity abutments and lateral cut-off wings. Briefly discussed are the theoretical stress analysis, structural model testing and construction plant. Described is the instrumentation for determining the actual behavior of the dam.

The dam was constructed in the years 1954-55 with the purpose of creating a reservoir on the Rio Freddo, in the Maritime Alps, having a net storage capacity of 325,000 cubic meters, to be used for daily and weekly regulation of the energy derived from the water of the Stura di Demonte basin in the Vinadio power station, which is owned by Soc. Edison.

Owing to the particular morphological structure of the valley at the site selected for the dam, double curvature thin arch structure was adopted, abutting directly on the rock up to a height corresponding to El. 1177 and on two concrete gravity thrust blocks for the upper portion, up to El. 1206 corresponding to crest level.

The portions of the valley section left free by the arch dam were filled in on the right bank by means of a solid gravity straight-axis wing and on the left bank by a spillway structure (Fig. 1).

The vault, i.e., the central arched part of the dam, has a maximum height of 40.50 m and thickness varying from 1.40 at the top to 5 m at the bottom. The horizontal arches are circular with single center for both intrados and extrados; consequently, thickness is constant from crown to abutments.

The length at the crest is of 120.90 m, and the chord corresponding to the extrados at the crest is about 100 m.

The principal geometrical data of the vault are:

– Vertical curvature radii of the crown section centerline	59.21 ÷ 118.42 m
– Horizontal extrados radii	42.52 ÷ 58.28 m
– Horizontal intrados radii	38.53 ÷ 55.78 m
– Angle measurements	40.00 ÷ 132.06 centesimal degrees
– Volume of the vault	10,000 m^3

Both facings of the vault are in concrete and steel forms were used for pouring. No special plaster was applied and the facings were provided with a light horizontal and vertical stress spreading reinforcement (Fig. 2).

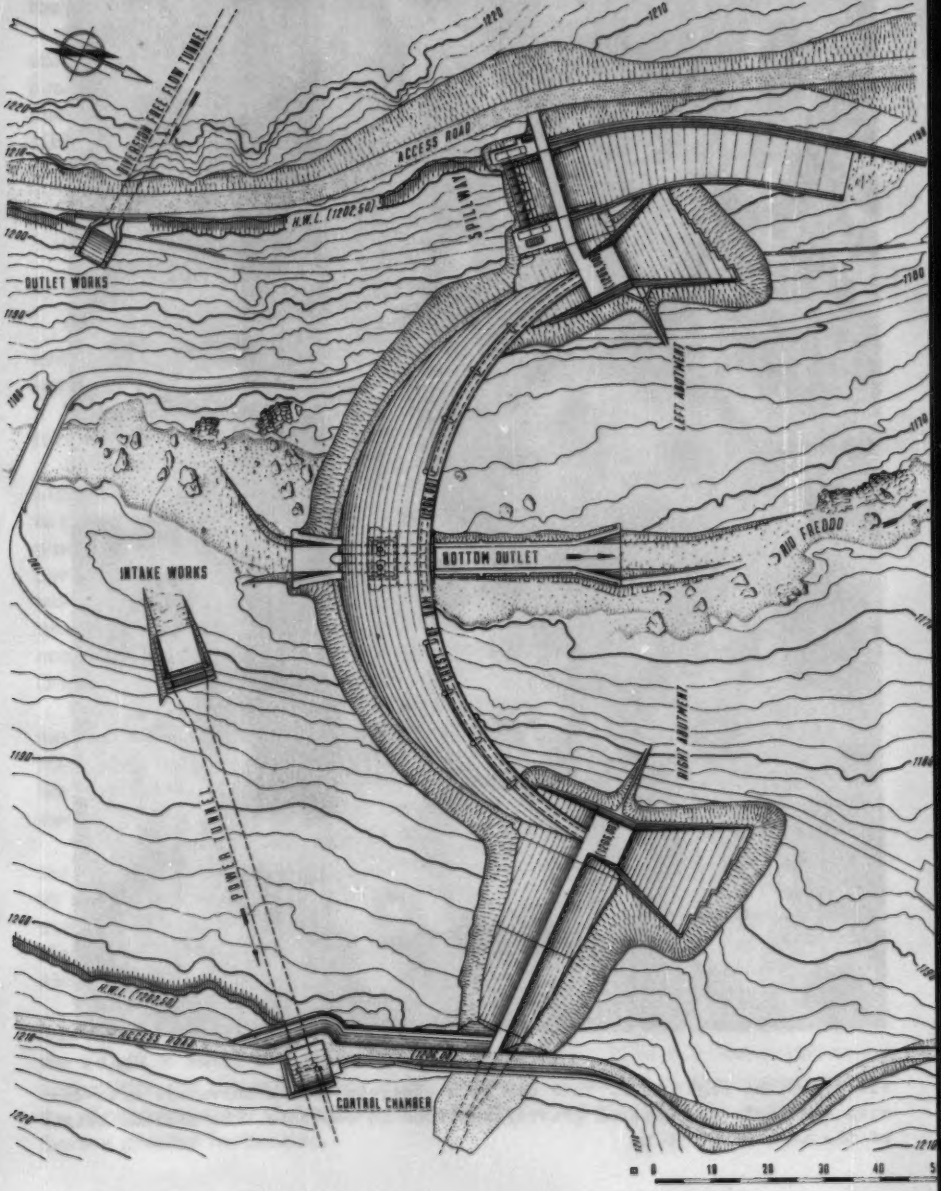
During construction, the vault was divided into 9 sections by means of 8 temporary vertical joints which were cement-grouted after shrinkage. Grout leakage is prevented by use of U-shaped 2 mm thick copper strips keyed on both sides into the concrete near the upstream and downstream facings along the joint.

The dam crest is established at El. 1204, which is the normal max. flood level; the max. exceptional flood level, which is expected to reach El. 1205, would cause the dam to overflow with a head of 1 m.

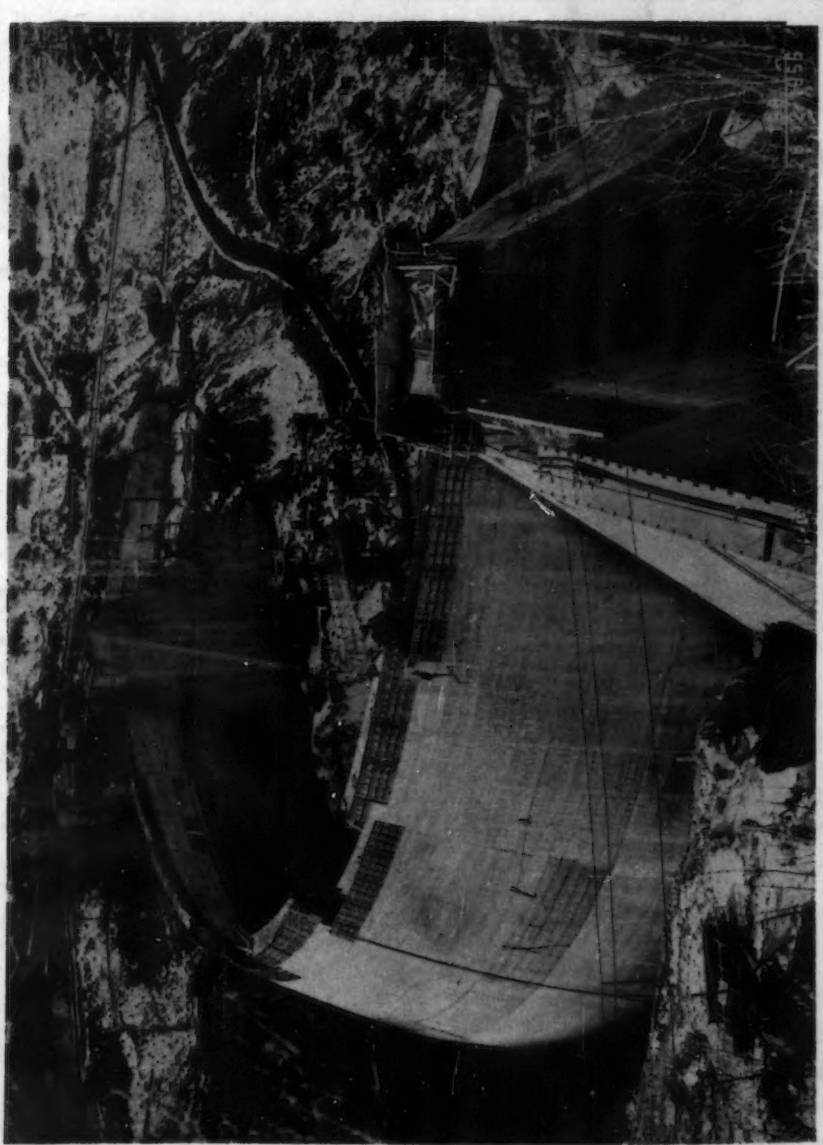
The exceptional overflow of the dam is allowed because of the considerable inclination of the vault towards downstream (about 9 m at the crown section)

FIG. 1

GENERAL LAYOUT



RIO FREDDO DAM FIG.2



View from the right bank. This dam is a concrete double-curvature arch-dam with abutments and lateral gravity sections in concrete. The maximum height of the arch if 40.50 m.

and of the peculiar shape of the cross-section of the crest arch, both of which features contribute to the discharge of the overflowing water at a sufficient distance from the foot of the dam (Fig. 3).

On the downstream face 4 horizontal inspection footbridges are arranged at El. 1196, 1187, 1178 and 1174 respectively.

The solid gravity shoulders, onto which the top portion of the vault is abutted, have a mean height of 27 m; the slope of the upstream face is 0.535, the slope of the downstream face 0.70 and the mean thickness 7.80 m.

Special care has been taken in designing the shoulders, with the purpose of ensuring at the same time a suitable abutment for the vault and good stability conditions in the shoulders themselves. The orientation of the axis of the shoulder was chosen to make the mean stress plane coincide as much as possible with the symmetrical plane of the shoulder.

The volume of each shoulder block is about 6,000 m^3.

Statical Analysis for the Vault and Shoulders

a) Vault

The vault has been considered as consisting of an assembly of independent arches, keyed at the abutments; these arches are subject to hydrostatic pressure, dead-weight and changes in temperature.

The effect on resistance of vertical elements the (cantilevers) was therefore not taken into account.

In order to make an imaginary subdivision of the vault for purposes of design, based on the results of recent experience carried out on models of arch dams, it was assumed that the isostatic lines of the structure lie on slanting planes instead of on horizontal ones which show an increase in dip with a corresponding decrease in elevation. Consequently the unitary eight arches which were being analyzed, are the ones resulting from the intersection of the intrados and extrados surfaces of the dam with planes belonging to a group coming from a horizontal axis normal to the plane of symmetry of the vault (Fig. 6).

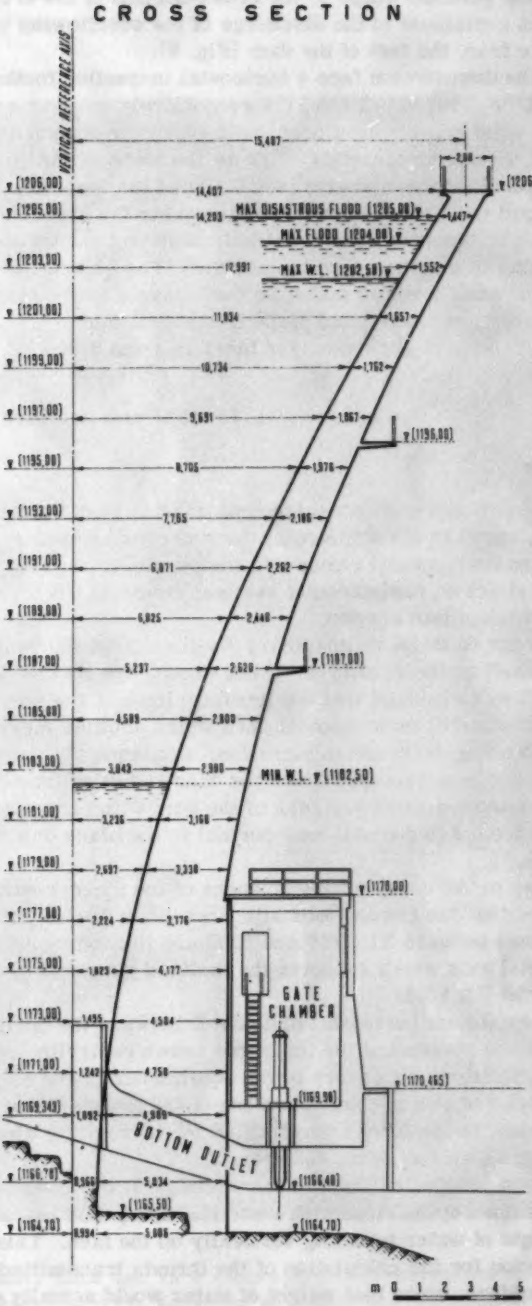
Owing to the considerable fineness of the upper portion of the vault, it was assumed that the related isostatic lines lie in horizontal planes; consequently the arches between El. 1206 and 1202 are limited by horizontal planes; the horizontal axis which supports the inclined planes determining lower arches was set at El. 1202.

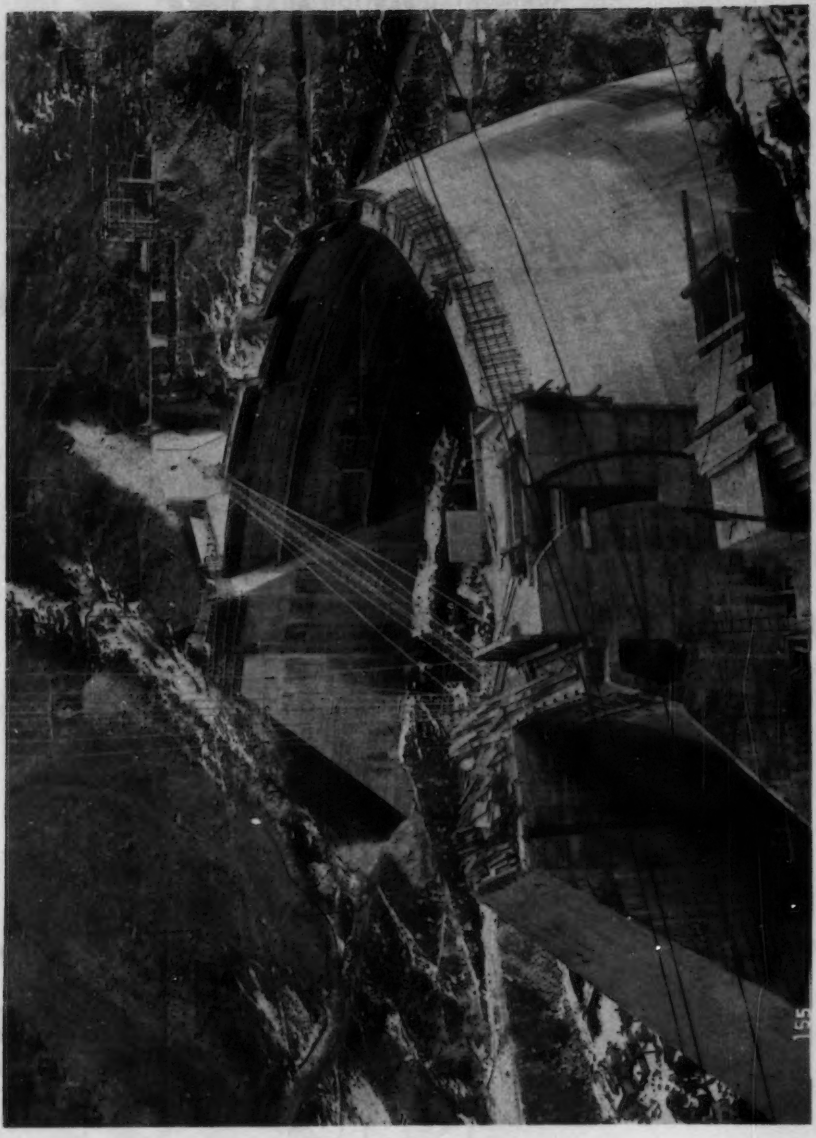
The minimum horizontal distance D between the horizontal axis supporting the inclined planes and the top of the crown centerline section was determined by the conditions necessary to the equilibrium of the elemental arches in every point of this section, each one of the arches being subject to hydrostatic load, to the forces transmitted by the sections above and below each element, as well as to its deadweight.

For the sole calculation of this distance D the upstream face of the dam was assumed cylindrical with a vertical axis, thus leaving out the effect of the weight of water pressing vertically on the face. This is a conservative assumption for the calculation of the thrusts transmitted by the arches onto the shoulders, since that weight of water would actually give the isostatic lines a greater inclination.

CROSS SECTION

FIG. 3

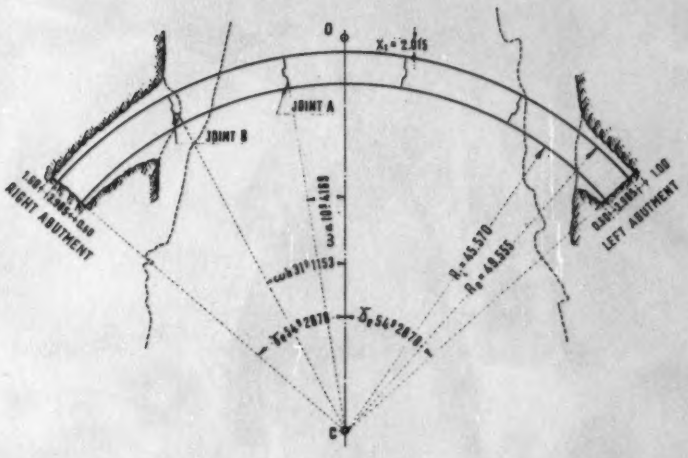




View from the left bank. In foreground see the gravity section with the spillway. The height of the abutments is about 27 m.

FIG. 5

HORIZONTAL SECTION AT EL. 1176.00



HORIZONTAL SECTION AT EL. 1188.00

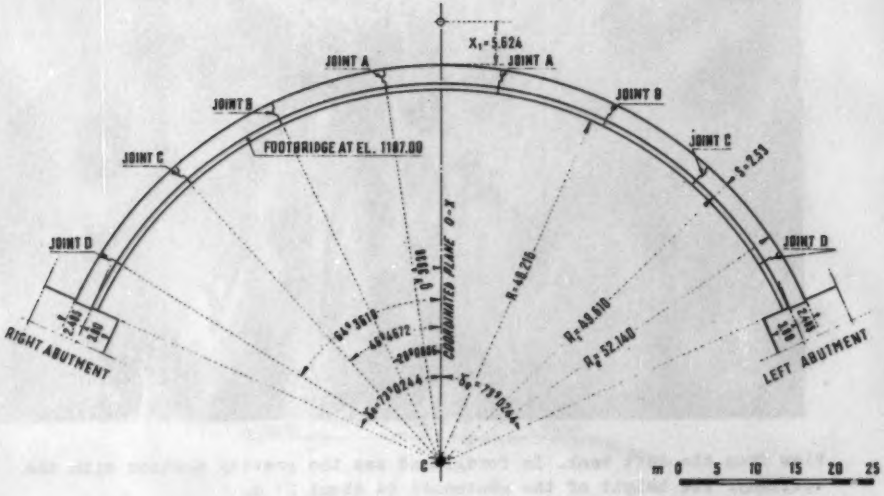
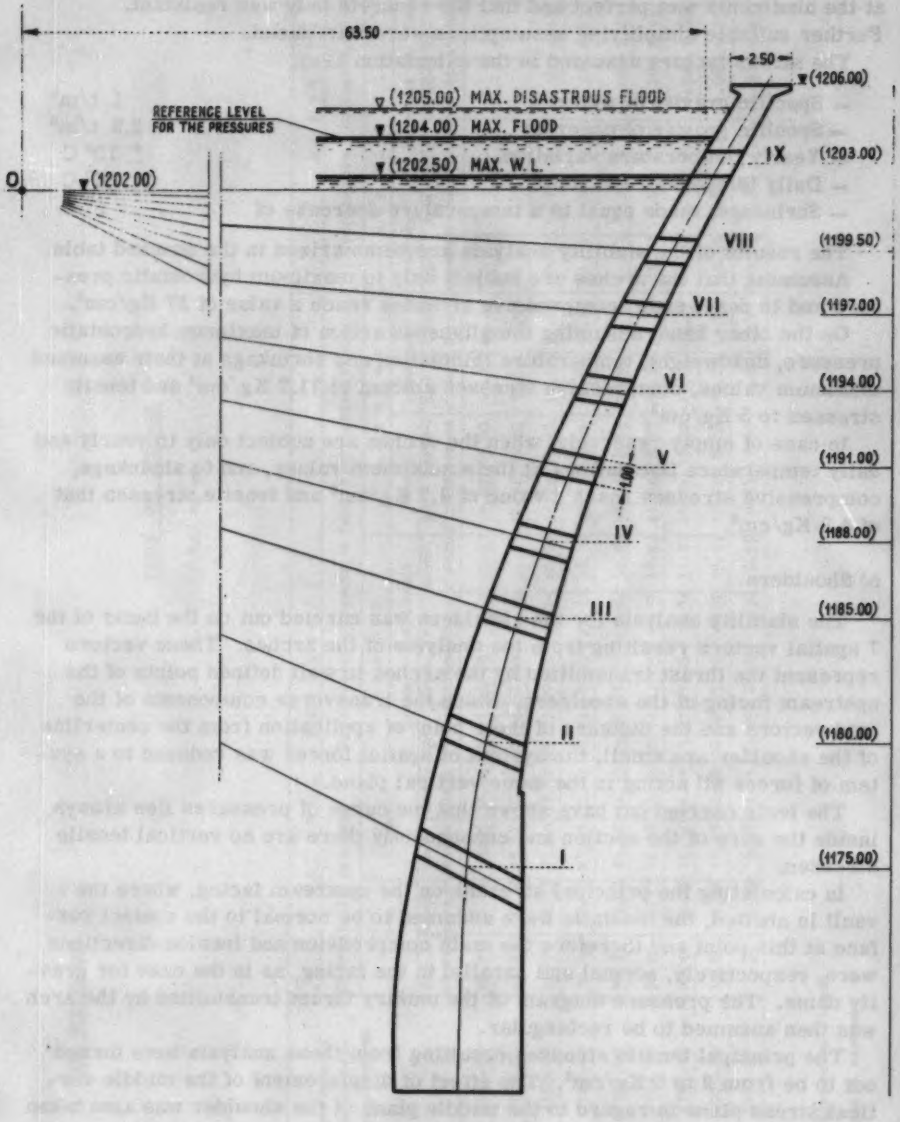


FIG. 6

SECTION AT THE CROWN WITH THE LINES OF THE PLANES DETERMINING THE CHECK ARCHES

1 : 200



Nine arches were analyzed, 8 of which dip; the one at El. 1203 is horizontal.

These arches were assumed to be subject to the entire hydrostatic pressure acting on each one of their points; this pressure therefore increases from crown to abutments for the arches below El. 1202.

The stability analysis of the nine arches was carried out in agreement with the principles of the theory of elasticity and with the assumptions that keying at the abutments was perfect and that the concrete only was resistant. Further suitable simplifying assumptions were introduced.

The stress factors assumed in the calculation are:

– Specific gravity of water	1 t/m³
– Specific gravity of concrete	2.5 t/m³
– Yearly temperature variation	± 10° C
– Daily temperature variation	± 6° C
– Shrinkage, made equal to a temperature decrease of	3° C

The results of the stability analysis are summarized in the annexed table.

Assuming that the arches are subject only to maximum hydrostatic pressure and to deadweight, compressive stresses reach a value of 37 Kg/cm².

On the other hand, assuming simultaneous action of maximum hydrostatic pressure, deadweight, temperature fluctuation and shrinkage at their assumed maximum values, compressive stresses amount to 71.7 Kg/cm² and tensile stresses to 5 Kg/cm².

In case of empty reservoir, when the arches are subject only to yearly and daily temperature fluctuations at their maximum values, and to shrinkage, compressive stresses reach a value of 4.7 Kg/cm² and tensile stresses that of 4.9 Kg/cm².

b) Shoulders

The stability analysis for the shoulders was carried out on the basis of the 7 spatial vectors resulting from the analysis of the arches. These vectors represent the thrust transmitted by the arches to well defined points of the upstream facing of the shoulders. Since the transverse components of the said vectors and the distance of their point of application from the centerline of the shoulder are small, the system of spatial forces was reduced to a system of forces all acting in the same vertical plane.

The tests carried out have shown that the curve of pressures lies always inside the core of the section and consequently there are no vertical tensile stresses.

In calculating the principal stresses on the upstream facing, where the vault is abutted, the isostatic were assumed to be normal to the contact surface at this point and therefore the main compression and tension directions were, respectively, normal and parallel to the facing, as is the case for gravity dams. The pressure diagram of the unitary thrust transmitted by the arch was then assumed to be rectangular.

The principal tensile stresses resulting from these analysis have turned out to be from 2 to 6 Kg/cm². The effect of displacement of the middle vertical stress plane in regard to the middle plane of the shoulder was also taken

RIO FREDDO DAM

TABLE OF STRESSES

Kg/ sq.cm

Elevation of arch center-line at crown	FULL RESERVOIR												EMPTY RESERVOIR							
	Hydrostatic pressure				Hydrostatic pressure + temp.fluctuation + shrink.								Temperature fluctuation + shrinkage							
	Crown		Abutment		Summer				Winter				Summer				Winter			
					Crown		Abutment		Crown		Abutment		Crown		Abutment		Crown		Abutment	
	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream	up-stream	down-stream
1175.00	44.95	8.51	8.37	36.93	43.27	8.88	3.50	41.43	53.83	-5.00	-0.22	44.41	-	-	-	-	-	-	-	-
1180.00	64.40	8.79	15.92	30.26	60.24	12.68	12.36	33.85	71.72	0.56	14.02	31.99	-	-	-	-	-	-	-	-
1185.00	34.94	44.53	23.90	37.17	29.45	49.96	20.15	40.90	41.91	37.42	25.35	35.68	-1.67	1.89	2.56	-2.49	3.15	-3.57	-4.86	4.73
1188.00	52.13	22.17	25.11	32.50	46.74	27.53	21.44	36.16	58.84	15.35	26.74	30.84	-1.51	1.70	2.36	-2.31	2.83	-3.16	-4.40	4.31
1191.00	50.65	21.16	25.18	31.35	45.62	26.15	21.62	34.89	56.78	14.95	26.96	29.55	-1.33	1.45	2.12	-2.09	2.43	-2.67	-3.90	3.83
1194.00	36.06	24.17	18.18	33.49	31.14	29.09	14.72	36.94	41.94	18.23	19.98	31.66	-1.17	1.27	2.02	-1.99	2.13	-2.29	-3.68	3.61
1197.00	20.43	23.99	11.54	28.61	15.67	28.76	8.34	31.81	26.03	18.32	13.32	26.81	-1.04	1.11	1.76	-1.74	1.88	-2.01	-3.18	3.14
1199.50	14.89	12.92	11.35	16.50	10.62	17.21	8.32	19.54	19.72	8.03	12.88	14.94	-0.69	0.74	1.90	-1.88	1.25	-1.34	-3.40	3.36
1203.00	3.94	3.44	3.08	4.21	-0.09	8.10	0.30	6.99	8.63	-1.90	4.60	2.67	-0.83	0.88	1.62	-1.60	1.49	-1.56	-2.88	2.84

\+ compressive stress

\- tensile stress

into consideration and the ensuing bending and twisting moments were also calculated. The relative stresses were found to be very small with maximum values of 0.3 and 1.25 Kg/cm² respectively.

Model Testing

To sustain and complete the calculation, tests were carried out at the I.S.M.E.S. institute of Bergamo on a 1:50 scale model made of a pumice cement mixture (Fig. 8).

The effect of deadweight in the shoulders was reproduced by means of steel tension cable. Hydrostatic pressure was obtained by means of hydraulic winches with suitable load spreaders.

Elastic tests and ultimate strength tests were made on the model and they showed that the dam acts as a double-curvature plate.

The elastic deformations measured (under a load about 4 times the maximum hydrostatic head) showed medium horizontal compressive stresses. The maximum values were 28.5 Kg/cm² at the upstream facing and 31 Kg/cm² at the downstream facing.

The vertical stresses at the upstream face, except for limited areas near the abutments, where small tensile stresses appear, were always compressive stresses.

At the downstream facing the stresses were on the whole tensile ones with a maximum of about 15 Kg/cm² in the same region where maximum horizontal compressive stresses also appear. However, if the effect of deadweight had been taken into consideration, these stresses would have been smaller.

The ultimate strength tests showed a practically linear elastic behaviour up to about 7 times the maximum hydrostatic head, while the first breaks on the downstream side were observed with a load of only 5.7 times the maximum one.

Load tests were then carried out as far as the test installation allowed (the load was almost 10 times the maximum hydrostatic head) and under this condition too the elasticity of the model was almost perfect.

The tests carried out on the shoulders showed very moderate stresses.

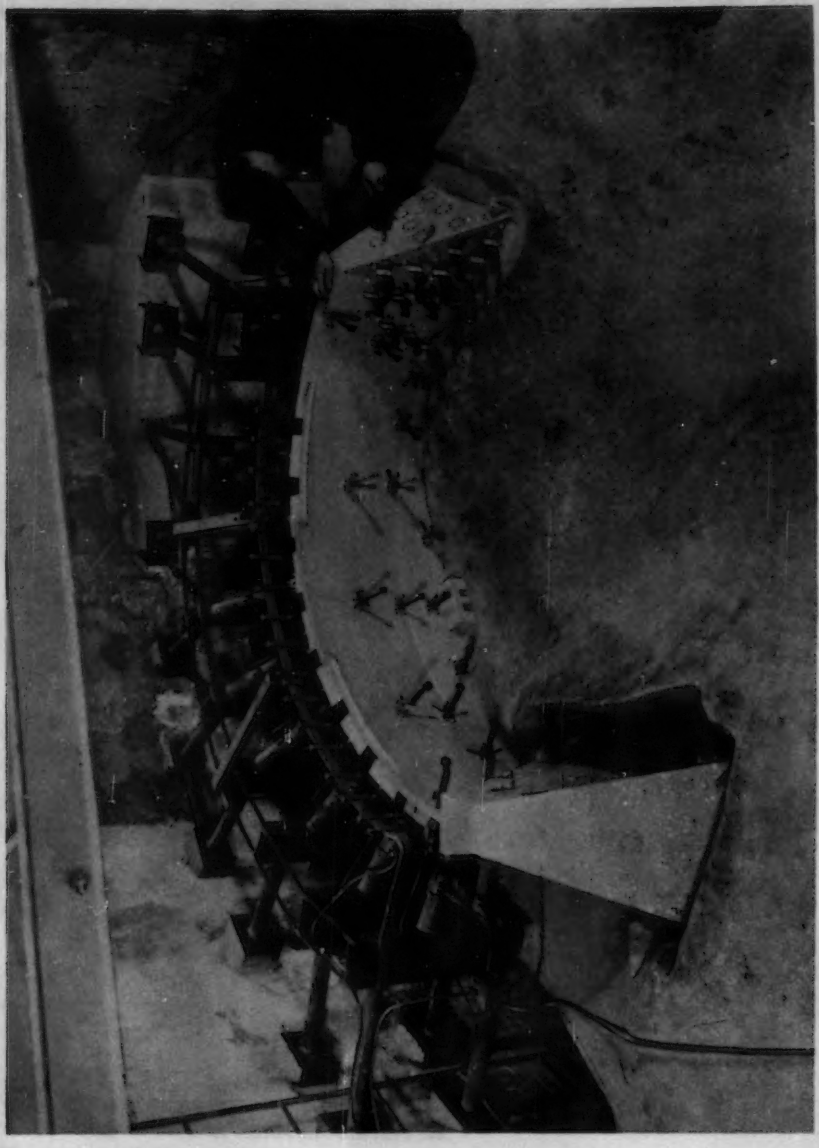
For the left shoulder, a maximum compression amounting to about 10 kg/cm² was recorded in the lower larger part, while the stresses at the upstream facing amounted to 2 kg/cm² on the inside (wet side) and 5 kg/cm² on the outside (dry side).

For the right shoulder the maximum stresses were 7.5 kg/cm² in the lower larger part and on the upstream facing approximately zero on the inside and 3 kg/cm² on the outside.

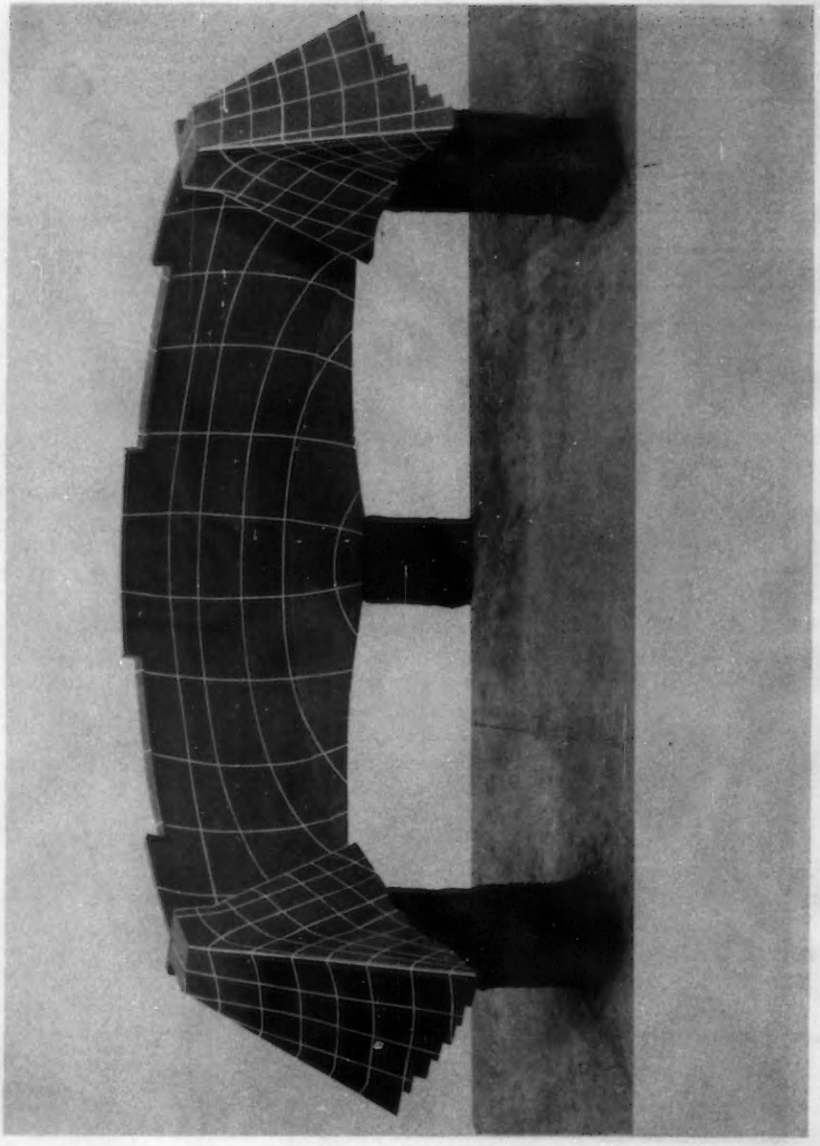
The isostatic lines plotted coincided sufficiently closely with those previously assumed as may be seen in Fig. 8.

Construction

The foundation excavations for the arch dam, shoulders and gravity wings were driven to a sufficient depth to make sure that the whole structure was founded on sound rock. Non-disruptive explosives were used and the



Testing model constructed on the scale 1:50 complete with the equipment for applying the load and with the instruments for measuring the deforma tions.



The isostatics on the downstream face of the arch-dam and on the abutments, as resulting from the load tests carried out on the model.

excavations were finished by hand using demolishing tools, without the use of explosives.

The aggregates for concrete were taken from borrow pits on the shore of the river Stura di Demonte; here they were sorted by means of rotary screens, after washing under pressure and the addition of material taken from the same borrow pit and suitably crushed.

The grading was: from 0 to 3 mm, from 3 to 10 mm, from 10 to 30 mm and from 30 to 60 mm.

The graded aggregates were then taken to the construction plant by truck, stored in bins and subsequently batched by weight.

Slow-setting cement was used, with standard mortar strength equal to 500 kg/cm^2 after 28 days.

The cement was batched by weight and proportioned in 300 Kg. per cubic metre for the vault and 200-250 Kg per cubic metre for the gravity wings.

Samples were taken daily from the concrete and submitted to strength tests. The tests were satisfactory having shown the following values at ultimate strength after a 7 day curing:

- 207 Kg/cm^2 for concrete at 250 Kg/m^3, with peak values of 277 and minimum values of 128
- 318 Kg/cm^2 for concrete at 300 Kg/m^3, with peak values of 417 and minimum values of 273.

The test samples taken after a 28 day curing gave instead the following figures:

- 266 Kg/cm^2 for concrete at 250 Kg/m^3, with peak values of 329 and minimum values of 177
- 372 Kg/cm^2 for concrete at 300 Kg/m^3, with peak values of 474 and minimum values of 314.

The concrete pouring was made by means of a main derrick with a 60 m boom placed in a central position so as to serve both the shoulders and the vault. For the right bank solid gravity wing an auxiliary derrick was used, with a 30 m boom (see Fig. 12).

The concrete was poured in steel forms and thoroughly acted on by vibrators.

Waterproofing grouting and bond grouting need no mention.

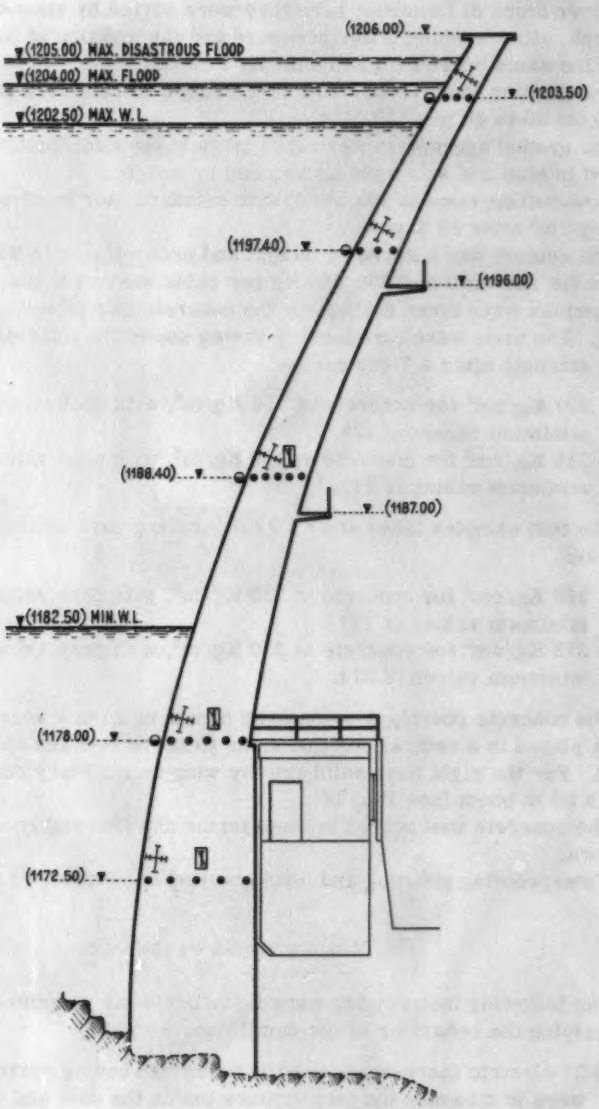
Measurements on the Dam

The following instruments were installed in the structure with the purpose of studying the behaviour of the dam (Figs. 9 and 10).

1) 31 electric thermometers with a central reading system, 27 of which were to measure the temperature inside the dam and 4 to measure water temperature at different elevations.
2) 90 electrically controlled thermal-extensometers were inclosed in the concrete about 40 cm from the upstream facing to measure temperatures and temperature-deformations. Five of these extensometers were isolated from the structure; they can detect both temperature and deformations due to thermal effect alone and to humidity.

CROSS SECTION
1 : 200

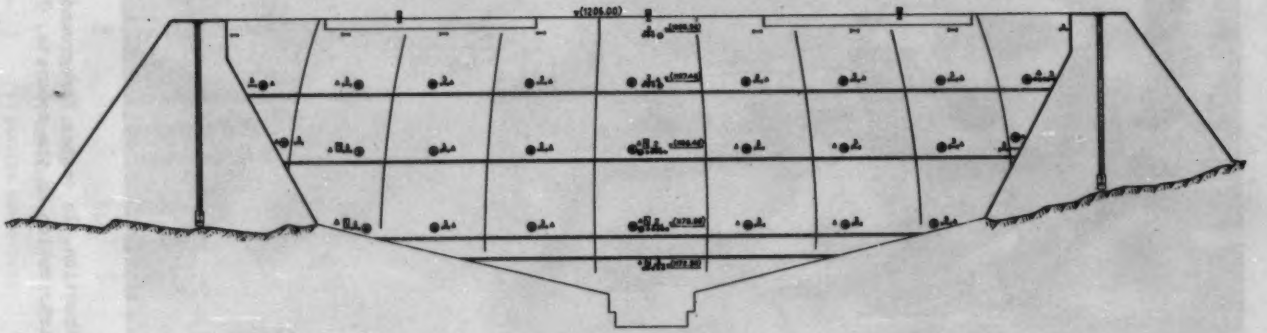
FIG. 9



- ELECTRICAL THERMOMETER
- ELECTRICAL THERMOMETER FOR WATER
- ELECTRICAL HUGGENBERGER EXTENSOMETER
- ISOLATED HUGGENBERGER EXTENSOMETER

POSITION OF MEASURING INSTRUMENTS

DOWNSTREAM FACE



- SET OF REMOVABLE EXTENSOMETERS
- ELECTRICAL EXTENSOMETER
- ISOLATED EXTENSOMETER
- TRIANGULATION TARGET
- ELECTRICAL THERMOMETER
- ELECTRICAL THERMOMETER FOR WATER
- ADJUSTABLE COLLIMATOR TARGET
- PENDULUM

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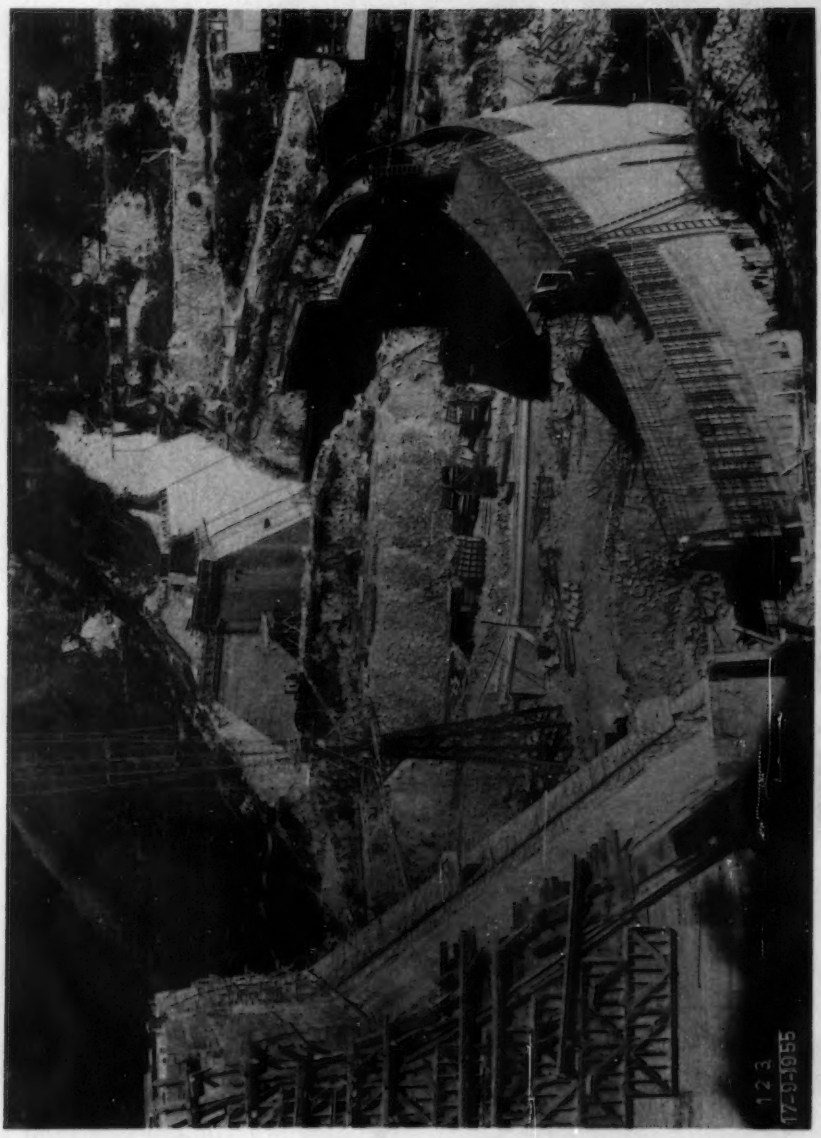
FIG. 10

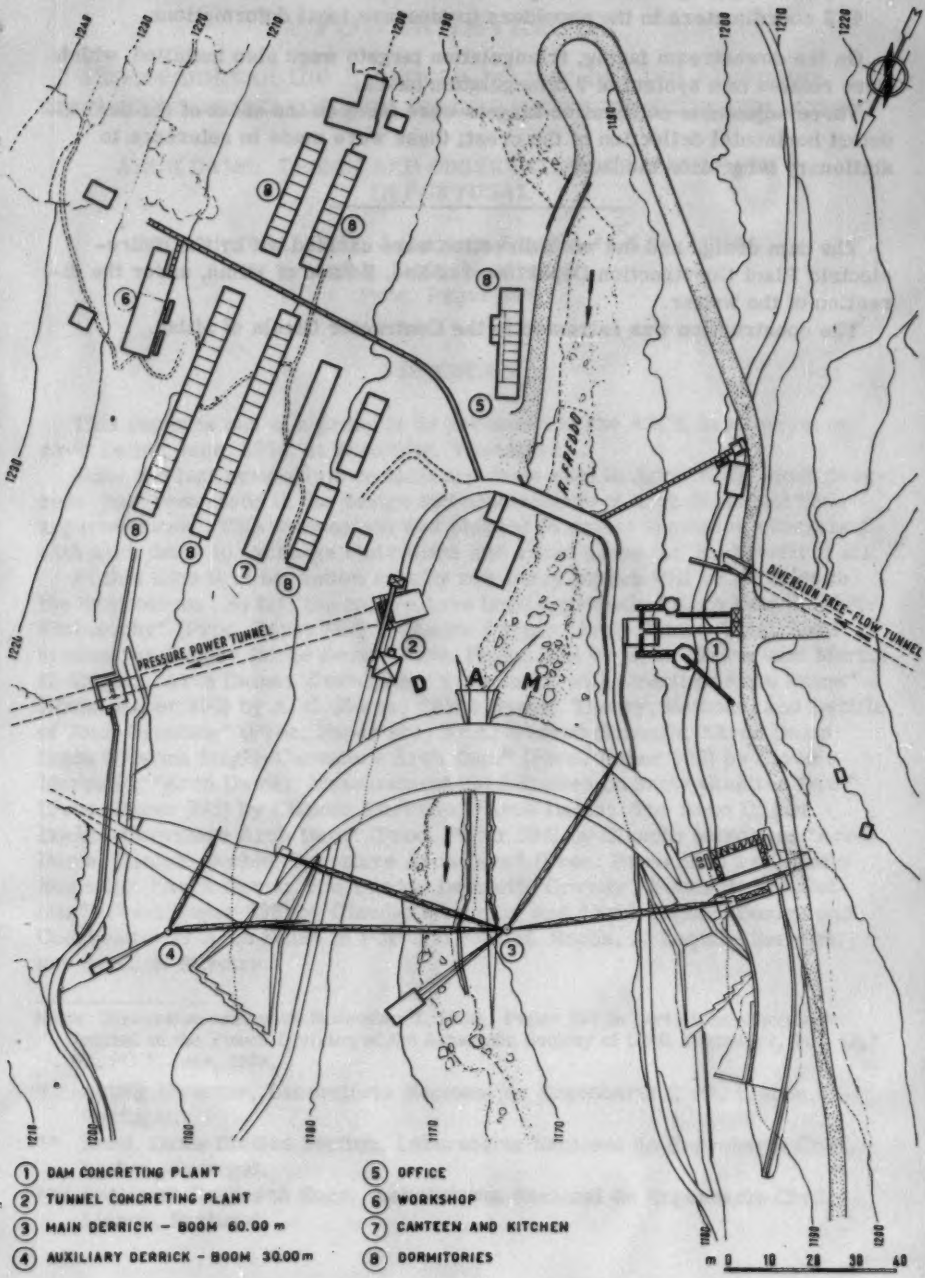
RIO FREDDO DAM FIG.11



View of the dam under construction. In center foreground see the main derrick with a 60 m boom for the distribution of the concrete. The concrete was poured in steel forms.

CONSTRUCTION PLANT
LAYOUT

FIG. 12



(1) DAM CONCRETING PLANT
(2) TUNNEL CONCRETING PLANT
(3) MAIN DERRICK – BOOM 60.00 m
(4) AUXILIARY DERRICK – BOOM 30.00 m
(5) OFFICE
(6) WORKSHOP
(7) CANTEEN AND KITCHEN
(8) DORMITORIES

RIO FREDDO DAM

3) 25 rosette-shaped bases for removable extensometers were installed on the downstream facing at points corresponding to the measurement point on the upstream facing.
4) 2 coordimeters in the shoulders to measure local deformations.

On the downstream facing, triangulation targets were also installed, which were related to a system of 9 triangulation bases.

Three adjustable collimation targets were fixed on the crest of the dam to detect horizontal deflection of the crest; these were made in reference to stationary targets on the banks.

The dam design and the work direction were carried out by the Hydroelectric Plant Construction Department of Soc. Edison of Milan, under the direction of the writer.

The construction was entrusted to the Contractor Girola of Milan.

Paper 997 PO 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DESIGN AND OBSERVATION OF ARCH DAMS IN PORTUGAL

M. Rocha,* M. ASCE, J. Laginha Serafim,** A.M. ASCE, and A. F. da Silveira***
(Proc. Paper 997)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams held in April,1939, much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be included in the Symposium. So far, ten papers have been approved: "Arch Dams: Their Philosophy" (Proc. Paper 959) by Andre Coyne; "Arch Dams: Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) by R. E. Glover and Merlin D. Copen; "Arch Dams: Portuguese Experience with Overflow Arch Dams" (Proc. Paper 990) by A. C. Xerez; "Arch Dams: Theory, Methods, and Details of Joint Grouting" (Proc. Paper 991) by A. Warren Simonds; "Arch Dams: Santa Giustina Singlê-Curvature Arch Dam" (Proc. Paper 992) by Claudio Marcello; "Arch Dams: Measurements and Studies on Santa Giustina Dam" (Proc. Paper 993) by Claudio Marcello; "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam" (Proc. Paper 994) by Claudio Marcello; "Arch Dams: Isolato Double-Curvature Arch Dam" (Proc. Paper 995) by Claudio Marcello; "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs" (Proc. Paper 996) by Claudio Marcello, and "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira.

Note: Discussion open until November 1, 1956. Paper 997 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

* Acting Director, Laboratoria Nacional de Engenharia Civil, Lisbon, Portugal.

** Head, Dams Studies Section, Laboratoria Nacional de Engenharia Civil, Lisbon, Portugal.

*** Assistant Research Engr., Laboratoria Nacional de Engenharia Civil, Lisbon, Portugal.

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

SYNOPSIS

Data are presented on design model testing and field observation of six arch dams ranging in height from 60 m to 135 m. This has enabled later designs to be modified to gain increased economy with safety. Model studies determined stress conditions at irregularities in foundation and around special structures in arch.

INTRODUCTION

The lack of mineral fuel in Portugal has lead to a large program of hydroelectric undertakings, which started in 1946. In view of the hydrologic regimen of the rivers being characterised by large flows in the winter and very small flows in the summer, it was necessary to create large reservoirs which call for high dams.

As so far narrow valleys have been available, in which the ratio between the length and the height has not been greater than four, and, as in the great majority of these sites there were rocks of suitable quality, arch dams have been constructed. The undoubted safety and economy of this type of dam has also contributed largely to its adoption, even in cases where the site has not appeared very favourable.

The designs of all the dams built have been accompanied by model studies and the detailed observation of their behaviour carried out by the Labóratorio Nacional de Engenharia Civil. This has made it possible to evaluate the methods for analysing arch dams and also to obtain fundamental data on the most suitable shapes. Thus, whilst the two first arch dams built are of the arch gravity type with a cylindrical upstream face, the latter ones are of the cupola type, that is, with a vertical curvature and varying radius arches.

Characteristics of the Dams and Their Foundations

This paper describes some of the most important studies carried out on the main Portuguese arch dams built since 1946 (Fig. 1), whose principal characteristics are given in Table I.

In the construction of these dams, use was made of Portuguese cements, similar to type II of the A.S.T.M. standards, and concretes having aggregate with a maximum dimension of 15 cm. They were built in separate blocks having radial joints with a maximum distance between the joints of 15 m. Lifts of one and a half or two meters were laid, with an interval between each lift of 4 to 5 days and high frequency vibrators were used. Only in the Castelo do Bode and Cabril dams was cold water and ice used in mixing the concrete. No artificial cooling of the concrete in place was done.

The foundation characteristics were surveyed geologically and by borings and galleries. However, in view of the importance of the deformability of the foundation on the behaviour of arch dams, the need was recognised of

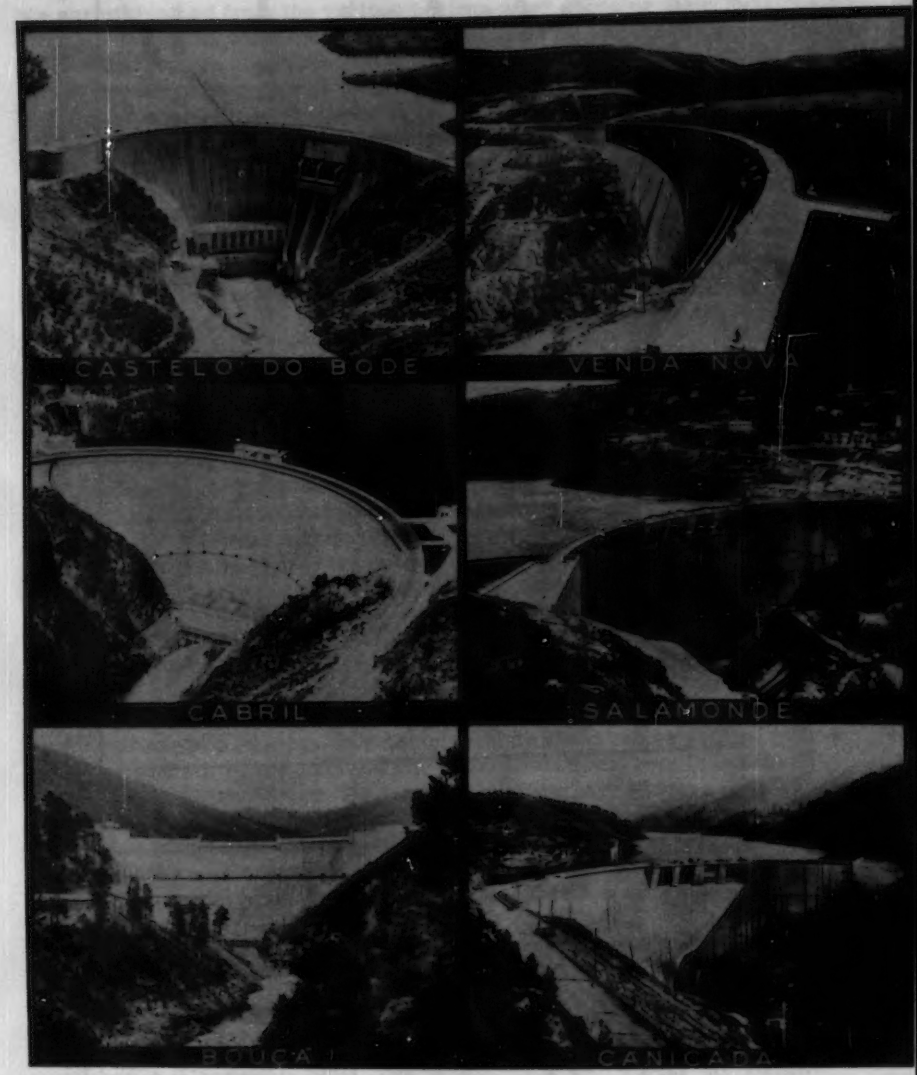


FIG. 1-Main Portuguese arch dams.

TABLE I

CHARACTERISTICS OF THE MAIN PORTUGUESE ARCH DAMS

Name of Dam (owner)	Type	Height (m)	Thickness at the base (m)	Upstream radius at the crest (m)	Angle at centre (degrees)	Volume of concrete (m^3)	Date of Construction		Designer
							Commencement of laying concrete	Conclusion	
Castelo do Bode (Hidro-Eléctrica do Zêzere)	AG,c.r.	115	34	150	112°	450.000	Jun.48	Aug.50	A. Coyne
Venda Nova (Hidro-Eléctrica do Cavado)	AG,c.r.	97	33	154,5	95°	220.000	Oct.49	May 51	A. Coyne
Cabril (Hidro-Eléctrica do Zêzere)	A, v.r.	135	19	155	110°	360.000	Jun.52	Dec.53	Owner
Salamonde (Hidro-Eléctrica do Cavado)	A, v.r.	75	8	115	100°	93.000	Apr.52	Apr.53	A. Coyne
Caniçada (Hidro-Eléctrica	A, v.r.	76	8	105	107°	80.000	Jul.53	Aug.54	Owner
Bouçã (Hidro-Eléctrica do Zêzere)	A, v.r.	60	7,5	95	102°50'	35.000	Nov.54	Aug.55	Owner

AG - arch-gravity; A-arch; c.r.- Constant radius; v.r.-variable radius

determining these characteristics both by laboratory and site tests. The laboratory tests consisted in the determination of the modulus of elasticity of the cores obtained from the various borings after choosing them judiciously and also of prisms from the galleries. The tests on site were carried out on all dam foundations mentioned here, except Venda Nova. The methods used in these tests and the results obtained have been described elsewhere.(1) One method consists in loading a section of a circular gallery with water under pressure and measuring the diameter variations. The other consists in loading simultaneously two opposite walls of a section of a gallery by means of jacks and metallic cushions filled with oil to distribute the load on the rock uniformly. These tests were, in many cases, carried out before and after grouting the rock round the galleries.

Table II shows the foundation characteristics of the dams described in this paper, and the values $\underline{Er}$ of the moduli of elasticity of the rock along the valley, deduced from all the tests carried out. It also shows the type and number of tests made.

Design of Arch Dams. Model Tests

As present day designs are based on the values of the stresses produced by the loadings, it becomes necessary, especially for structures involving great responsibility and cost, to seek methods which give the value of these stresses with reasonable accuracy.

When the construction of large arch dams was begun in Portugal, there was already some experience of model studies obtained from model tests of Santa Luzia Dam.(2) One criticism that could be made then of the current analytical methods for dam design was that they were either insufficient or very complex. It was concluded that, among the existing methods, the "trial load" method was the one which could supply the most reliable results, even though its full application, that is, with radial, tangential and twist adjustments, was very laborious and lengthy. In principle, this method can be applied to concrete dams of any shape and take into consideration the true foundation deformability. However, when the valley is considerably assymmetric or the foundation profile irregular, or when the dam has important gravity abutments or large openings—for example, spillway openings—or even when the foundations are heterogeneous, then the number of arches and cantilevers which have to be considered, as well as the number of trials, entail excessive work and time.

Besides this, whenever the hypotheses of Strength of Materials are inadequate, as happens in the fields of stress which exist round very irregular shapes, the method, by its very nature, cannot give satisfactory results.

In any case, the method was applied to all the dams mentioned in this paper, but in its abridged form, that is with radial adjustment of the arch and cantilever displacements only. The comparisons which were made between the results obtained by this abridged method and the results from models and observation(3) led to the conclusion that it can only be used for an estimate of the stresses in a preliminary design. As a rule, in the case of regular shapes, the stresses determined in the arches by this method are from ten to twenty per cent greater than those obtained from models, whilst for stresses at the base cantilevers the former stresses are much greater than the latter. It should also be noted that the maximum arch compressive stresses given by

TABLE II

CHARACTERISTICS OF THE FOUNDATIONS OF THE MAIN PORTUGUESE ARCH DAMS

Dam			Castelo do Bode	Venda Nova	Cabril	Salamonde	Caniçada	Bouçã
Right bank	Upper part	Rock	Altered schist	Schist (on top)	Granite	Granite	Granite	Altered schist
		Er ($kgcm^{-2}$)	40.000	40.000	150.000	80.000	100.000	20.000
	Lower part	Rock	Cristal. schist	Granite	Hard granite	Hard granite	Hard granite	Schist
		Er ($kgcm^{-2}$)	110.000	100.000	200.000	200.000	200.000	60.000
Bottom of the valley		Rock	Cristal. schist	Hard granite	Hard granite	Hard granite	Hard granite	Schist granite
		Er ($kgcm^{-2}$)	110.000	200.000	220.000	200.000	200.000	100.000
Left bank	Lower part	Rock	Cristal. schist	Hard granite	Hard granite	Granite	Hard granite	Granite
		Er ($kgcm^{-2}$)	110.000	200.000	200.000	80.000	150.000	100.000
	Upper part	Rock	Hard cristal. schist	Altered granite (on top)	Granite	Altered granite	Altered granite	Altered granite
		Er ($kgcm^{-2}$)	200.000	50.000	70.000 and 150.000 (upper)	20.000	50.000	20.000
Tests Carried out		In the Laboratory	41 cores 12 prisms	73 cores 10 prisms	78 cores 16 prisms	30 cores 11 prisms	55 cores 6 prisms	78 cores 34 prisms
		On Site	1 shaft with water	—	5 galleries with jacks	5 galleries with jacks 2 galleries with water	2 galleries with jacks	2 galleries with jacks

calculation and by the models, even when being of the same magnitude, are not found to be at the same points. The radial displacements—the only ones which are determined—are also from ten to twenty per cent greater than those of the models.

These results led to the use of model tests as the basis for the design of the Portuguese dams. As a matter of fact, the improvement in test techniques not only made it possible to be confident of the experimental results but also to reduce the time of test, which is now less than that needed for the use of the abridged "trial load" method. Thus, in the majority of the dams, the final design was based on the successive studies of models in which the shapes were progressively improved. These studies have been started as soon as the preliminary design of the structure was made. The test techniques used at present have already been described.(3,4) Table III shows the number and characteristics of the models, the duration of the tests and the cost of the model tests. As can be seen from this Table it is the practice at present to test at least two equal models symultaneously for each shape of dam analysed. By this means the reliability of the results is increased and the accuracy is improved by taking the mean values of the two models.

In model studies, as in calculations, it is assumed that the dam is a homogeneous, isotropic, elastic and continuous solid supported on an elastic foundation.

Normally the models are only analysed for hydrostatic pressure but in some cases the stresses due to the weight of parts of the structure itself were also determined. Usually the stresses due to this loading are calculated by assuming that the dam is built in separate blocks. The effects of the remaining loadings, such as temperature, earthquakes and uplift, are also calculated analytically and added to the stresses due to hydrostatic pressure. It should be noted that usually the stresses due to hydrostatic pressure are very much greater than the stresses due to the other loadings together.

The stresses in the models are determined by electrical strain gauges, the SR-4 Baldwin types A7 and A8 being particularly used.

Previously the isostatics of the two faces are determined with brittle lacquers. Radial, tangential and vertical displacements are also measured.

The measurements are nearly always determined for various water levels in the reservoir which is particularly important for their comparison with the prototype observations. This fact is a further advantage of the models over the calculation methods.

Recently a supplementary experimental method was investigated in the search for the best shapes of arch dams. It consists in determining the shapes taken by a rubber membrane fixed across a valley and loaded from downstream with water and with upward forces proportional to the weight of the concrete.(5)

The Value of Observations

The observation of dams both during and after construction has two objectives, the control of safety and the investigation of the behaviour. The observation can check the methods used in design, whether analytical or experimental, and also judge the building methods followed.

For the task of observation to be fruitful the equipment must be carefully chosen and the work of installing and reading the instruments, calculating and

TABLE III

MODELS OF MAIN PORTUGUESE ARCH DAMS

Name	Studies carried out	No. of models tested	Scale of the models	Material of the model and loading system	Duration of the tests (months)	Cost of the tests (dollars)
Castelo do Bode	Study I	1	1/ 75	Pd — M	8	4,800
	Study II	4	1/500	Pd — J	4	
Venda Nova	Study I	1	1/100	Pd — M	8	6,000
	Study II	2	1/300	Pd — J	4	
	Study III	2	1/ 30	Pd — J	3	
Cabril	Study I	2	1/300	Pd — M	6	7,800
	Study II	2	1/300	Pd — M	4	
	Study III	2	1/300	Pd — M	4	
	Study IV	1	1/400	A — M	6	
	Study V	1	1/400	A — M	3	
Salamonde	Study I	2	1/200	Pd — M	6	7,200
	Study II	2	1/200	Pd — M	4	
	Study III	2	1/200	Pd — M	4	
	Study IV	2*	1/200	Pd — M	4	
Caniçada	Study I	2	1/200	Pd — M	6	4,200
	Study II	2	1/200	Pd — M		
Bouçã	Study I	3	1/200	Pd — M	6	3,000

Notes

Pd - Mixture of plaster of Paris and diatomite.

A - "Alkathene".

M - Mercury contained in a rubber bag.

J - Set of jacks.

* - A" Marco" model was also built (Scale 1/250).

interpreting the results has to be carried out by fully trained specialist teams supported by a laboratory where the various tests both of the instruments and of the materials of the dam can be made.

However, in comparing the results of observation of a structure with the results of model tests a difficulty arises from the fact that the prototype is subject to many actions which are not easily reproducible in the model, such as temperature variations either from external variations or from dissipation of the heat of hydration of the cement, the anelasticity of the concrete and rock, foundation settlements, movements of the reservoir banks etc.

The Laboratorio Nacional de Engenharia Civil carries out[6,7] all the observation work in which other Government departments and the owners of the dams take part. An elaborate plan for observation is drawn up by the L.N.E.C. which also maintains a team on the site for placing and reading apparatus. The readings are sent regularly to the Laboratory where their calculations, plotting and interpretations are carried out, which calls for numerous tests on the concrete.

The values which have been observed so far are: absolute dislacements of points of the dam and on the ground by geodetic and alignment methods;[8] vertical displacements by precision levelling of the crest of the dam and at points on the rock downstream;[9] relative displacements by pendulums;[10] rotations with clinometers; joint movements, inside with joint meters and outside with extensometers;[11] air, concrete and reservoir water temperatures by thermo-electric couples; strains by means of strain meters;[12] compressive stresses with Carlson stress-meters; neutral pressures by Carlson meters; humidity with humidity measuring instruments, and uplift pressures with pressure tubes equiped with manometers.

Table IV gives the apparatus used in the arch dams referred to in this paper.

Results of Studies

Castelo do Bode

The Castelo do Bode dam (Figs. 1 and 2 and Table I) presents some important features. In the first place the right bank is a very deformable promontory which, in the upper part, supports a large artificial abutment. Though it is approximately symmetrical, the dam has two flood spillway openings of 15 x 12 m. Above these openings the dam was thickened in order to transmit the forces from one side of the opening to the other more effectively. The dam was calculated by radial adjustments between four arches and three cantilevers.

Model studies were undertaken to determine the general field of stresses, the most suitable dimensions for the thickening of the dam in the neighbourhood of the spillway openings and the dimensions of the artificial abutment. A first model was built (Table III) to a scale of 1/75 in which the various mechanical properties of the foundation were reproduced. The failure of this model, which occured in the third load test, showed the advisability of increasing the dimensions of the artificial abutment.

In order to expedite the construction and drying, four smaller models were built later to a scale of 1/500. These models were loaded with a system of 26 hydraulic jacks of different diameters.

The results of the tests (Fig. 3) showed that in the spillway area the

TABLES IV

APPARATUS PLACED IN PORTUGUESE ARCH DAMS

	Dams	Castelo do Bode	Venda Nova	Sala-monde	Cabril	Caniçada	Bouça
Displacements by geodatic method	Theodolite stations	4	3	4	4	4	4
	Sighting marks	17	18	14	19	14	14
	Check points	4	5	4	5	4	5
	Alignments	4	-	6	3	6	6
	Levelling points	25	15	10	21	10	10
Other Observations	Plumb-wires	3	3	1	2	1	-
	Coordinometer bases	-	-	-	-	-	-
	Coord.-Clin. bases	20	-	6	6	6	-
	Clinometer bases	3	3	14	14	14	10
	Strain--gauges	164 (a)	181 (a)	140 (b)	189 (c)	184 (c)	179 (c)
	Carlson stress-meters	-	-	-	-	6	9
	Resistance thermometers	71	80	-	-	-	-
	Thermo-electric couples	44	53	50	50	26	33
	Uplift points	-	-	-	12	-	-
	Carlson pressure meters	-	-	-	-	6	6
	Joints. Detachable gauges	151	55	43	100	50	50
	Joint-meters	-	-	21 (b)	55 (c)	21 (c)	49 (c)
	Moisture content meters	-	-	-	6	6	6
	Length of cables (m)	15.000	8.000	3.000	7.000	6.000	11.500

(a) Telemac (b) Galileo (c) Carlson

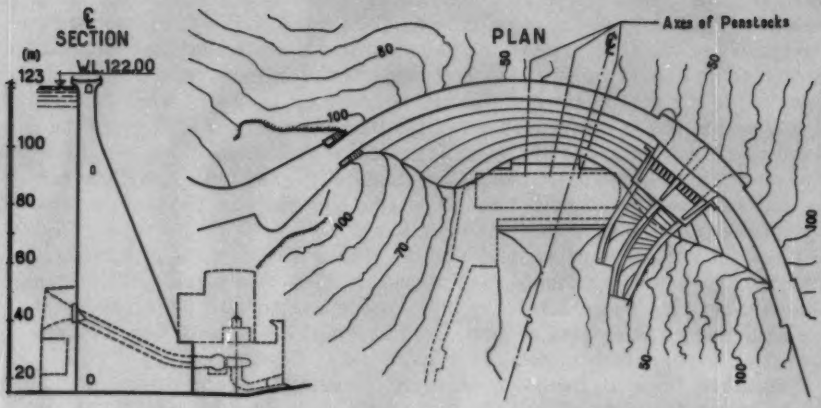


FIG. 2 – CASTELO DO BODE DAM. Plan and section.

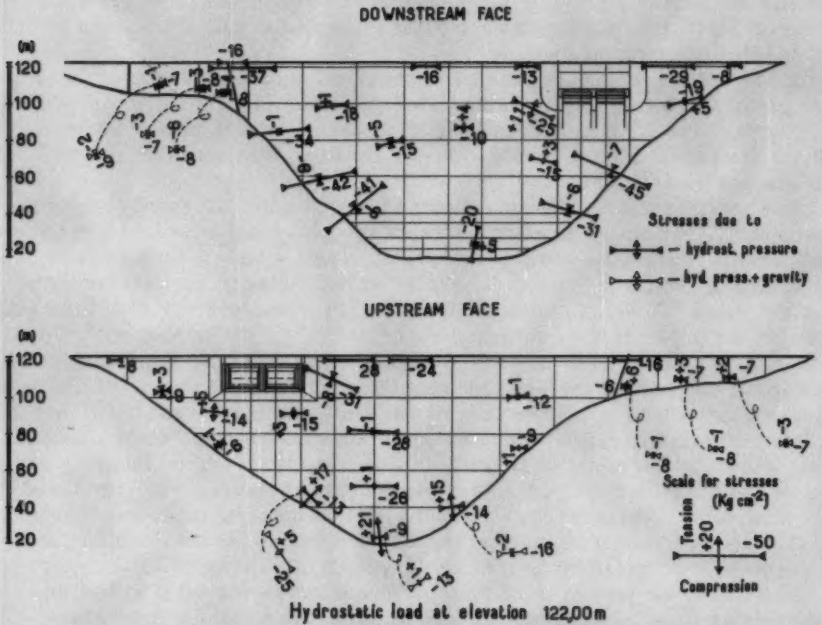


FIG. 3 – CASTELO DO BODE DAM. Stresses from model studies.

directions of the principal stresses were diverted by the openings made in the dam. The compressive stresses due to hydrostatic pressure did not exceed 45 $kgcm^{-2}$ on the upstream face or 28 $kgcm^{-2}$ on the downstream face. For these regions the calculations gave maximum horizontal stresses, in the arches, of 42 and 43 $kgcm^{-2}$ respectively. The maximum tensile stresses observed in the heel of the dam for the hydrostatic pressure were 21 $kgcm^{-2}$.

For points in the upstream face of the dam base, Fig. 3 also gives the sum of the stresses due to the hydrostatic pressure with those of the weight of the dam itself calculated analytically. The tensile stresses due to hydrostatic pressure are practically anulled. The same happens with the tensile stresses in the base of the abutment of the right bank.

As to the spillway openings, it was concluded that, in spite of their large dimensions, they only influence the stresses of the dam locally, the continuity being assured by the arch passing above these openings. The thickness of this arch should have been reduced as, in view of its rigidity, the stresses developed in its neighbourhood were high.

The model tests also made it possible to design the reinforcement to be placed inside the vertical walls of the openings in order to support the high tensile stresses which are developed there.(4)

Geodetic observations of the dam displacements (Fig. 4) show a movement of the whole dam toward the right bank and important movements of the abutment which reach a value of 2 cm. These displacements are a result of hydrostatic pressure on the actual promontory supporting the dam and also of the low modulus of elasticity of the rock in this region (about 40,000 $kgcm^{-2}$). However, as the displacements are being assymptotic with time it can be concluded that the structure is safe.

It was noted that the displacements observed in the upper part of the dam are greatly influenced by air temperature variations.

Fig. 5 shows some typical results of observations with strain meters. These strain meters are arranged in 45° rosettes of four meters and no-stress meters.(13)

The analysis of the diagrams shows a considerable difference in their development. The diagrams of downstream groups 2d and 3d accompany the air temperature variations showing compression with a rise in temperature. The diagrams of groups 1u, 2u and 3u, placed at one meter from upstream face, and the group 1d, which is protected by the power house, show very little influence of temperature variations.

Group 1u, during 1949, registered a continuous increase of negative strains due to the increase of the weight during construction. At the end of that year there was a reduction of the compression due to temperature fall and then a further increase in negative strain due to temperature rise. Since the reservoir was filled the effect of air temperature variations was no longer apparent on the behaviour of the diagrams and the strain meters only registered slight effects of water level variations and of the general cooling of the dam.

The group 1d never registered large strain variations but the effect of reservoir level variations is noticeable, principally in the vertical meter.

The annual air temperature cycle, whose effect is recorded without any appreciable time lag in groups 2d and 3d, produces a state of hydrostatic stress as is shown by the parallel development of the diagrams of the 45° meters in group 2d or of the vertical and horizontal ones in group 3d. These directions are those least affected in the two groups by variation of hydrostatic pressure. On the other hand the diagrams of the vertical and horizontal

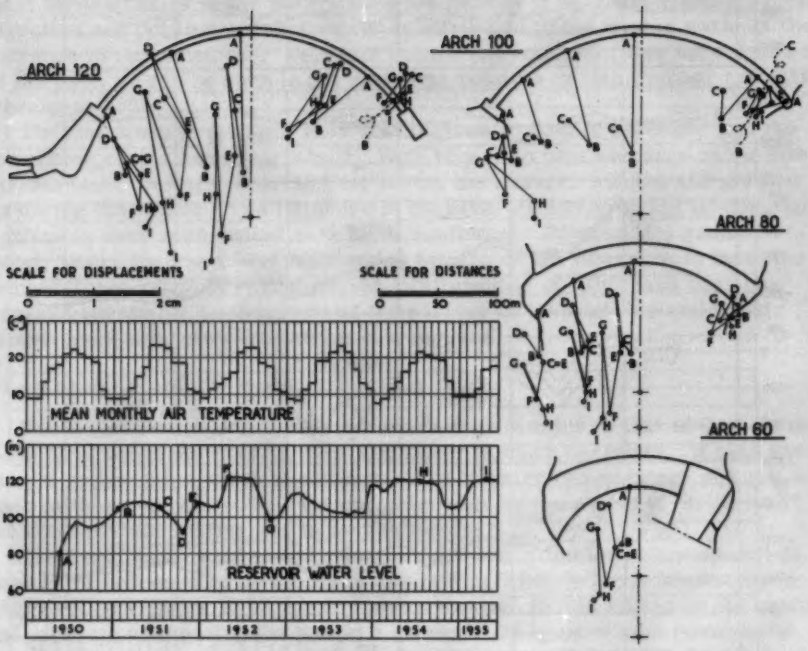


FIG. 4 - CASTELO DO BODE DAM. Horizontal displacements observed by the Geodetic Method.

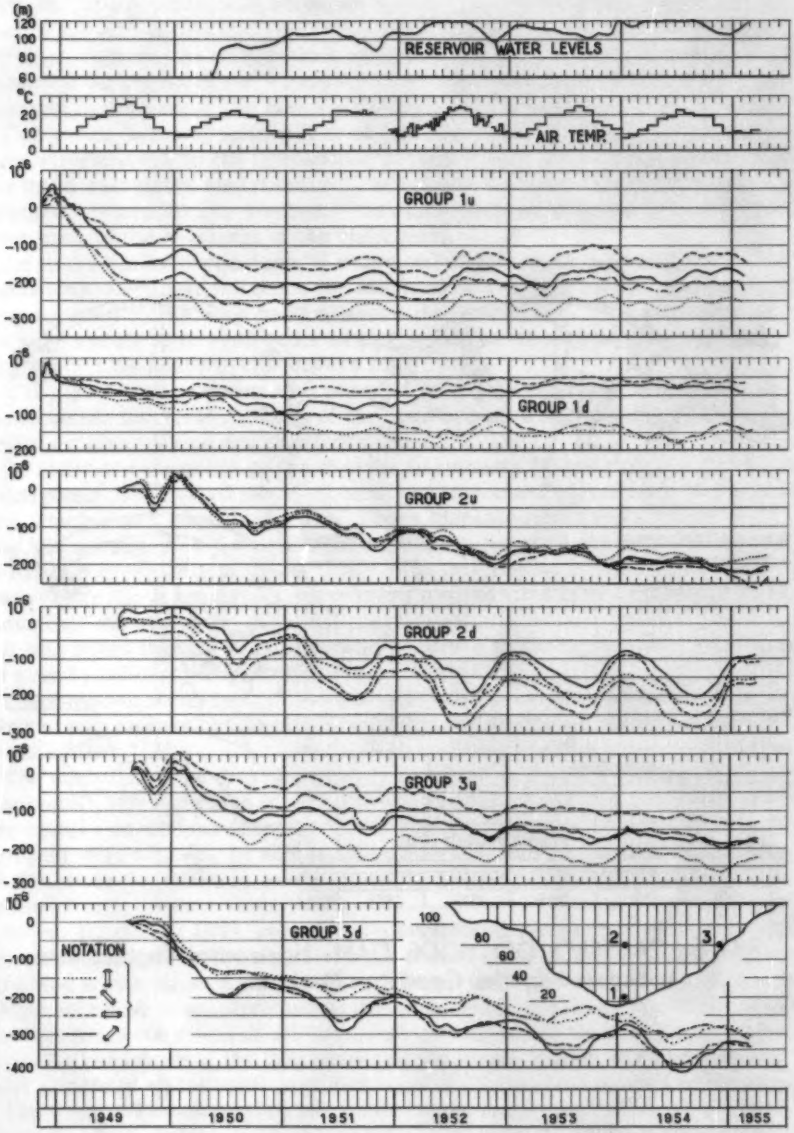


FIG. 5 – CASTELO DO BODE DAM. Corrected observed strains at points 1m from the upstream and downstream faces.

meters of group 2d and those of 45° in group 3d, which are those most affected by the variation in level of the reservoir, approach one another and separate with such variations. The same can be said in relation to group 3u although in this case the variations in strain due to temperature are small.

The isotherms and flow lines in Castelo do Bode dam and in the reservoir water are plotted in Fig. 6 for four typical dates of 1954 when the heat of hydration of the cement had already dissipated. The mean monthly air temperatures (T_A) of the month prior to the day in which readings were taken are given.

The analysis of the figure shows that the reservoir water still undergoes appreciable temperature variations to a depth of about 50 m. On the other hand the isotherms of the water round the intakes (Fig. 2 and 6) change their direction and became vertical, which is attributed to the flow of water at the entrance to the penstocks. This fact causes unusual heat flows near the heel of the dam, as can be seen in the diagrams relative to 30th October and 30th December 1954.

The flow lines in the dam show that it gives up heat to the water and the foundation rock all the year round. With regard to heat exchange at the downstream face, the dam receives heat during the summer months and gives it up during the winter. The influence of the solar radiation on the downstream surface is very accentuated as it faces southwest. Particularly round the crest, where the sun's heat falls on the top and on the downstream face, the temperatures reach very high values. Differences of 12°C have been observed between the temperature at the surface of the concrete and the air temperature one meter away from this surface.

Venda Nova

Venda Nova dam (Fig. 1) is a structure very similar to that of Castelo do Bode except that the lower part of the valley is much narrower. It also has a lateral spillway for which it was necessary to leave two openings of 8 x 6 m each in the structure. The dam was calculated by the abridged "trial load" method with four arches and three cantilevers.

The first model studied had a projecting rock in the lower upstream left bank as is represented by a broken line in Fig. 7 (Study I). A tension due to hydrostatic pressure of 43 $kgcm^{-2}$ was measured in this region on the model and after superimposing the weight a stress of 32 $kgcm^{-2}$ still remained. For this reason the canyon profile was made more regular by cutting the projecting rock and Study II was carried out in which tensile stresses of 15 $kgcm^{-2}$ were derived for the combined effect of hydrostatic pressure and weight. It should be mentioned that the studies on models were begun when construction was already started and therefore with the foundation opened up, that is when it was not possible to make large modifications in the design. It was therefore decided to make a joint ending in a gallery in the upstream face (Fig. 7, Study III). The object of this joint was to prevent the dam from cracking which would endanger its water-tightness. Once the joint opens the stresses due to the weight of the dam compensate for the tensile stresses due to the hydrostatic pressure at the inside end of the joint. In models of this solution (Study III) the stresses at the bottom of this joint and at various points on the two faces of the dam were determined. The tensile stresses in the middle region were in fact anulled by the joint but near the ends of the joint there were still tensions of 13 $kgcm^{-2}$ for the hydrostatic pressure and the weight.

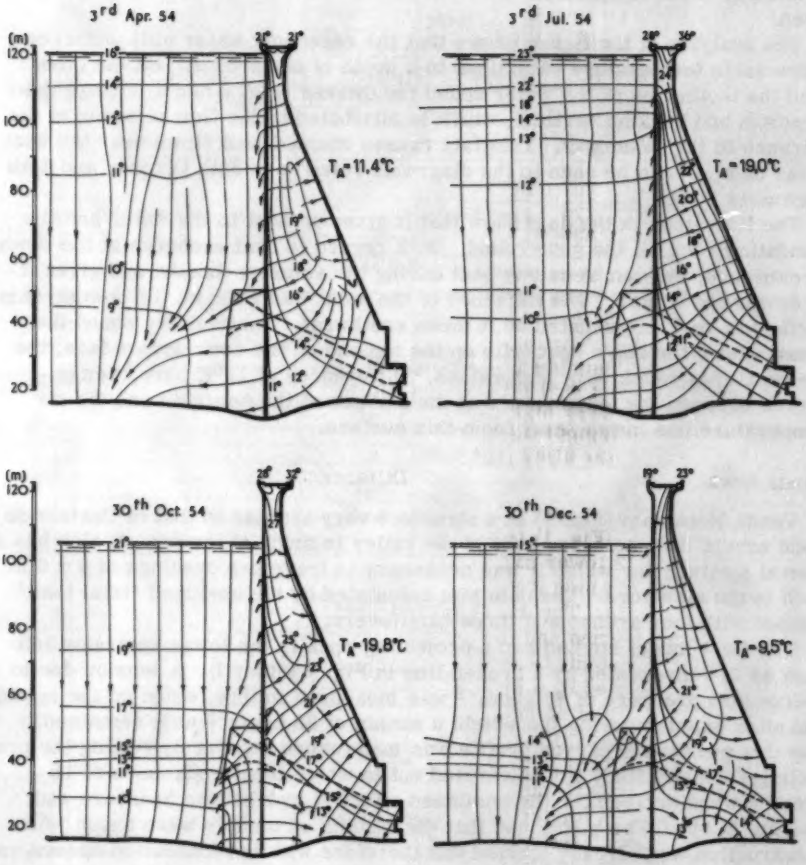


FIG. 6 – CASTELO DO BODE DAM. Flow lines and isotherms.

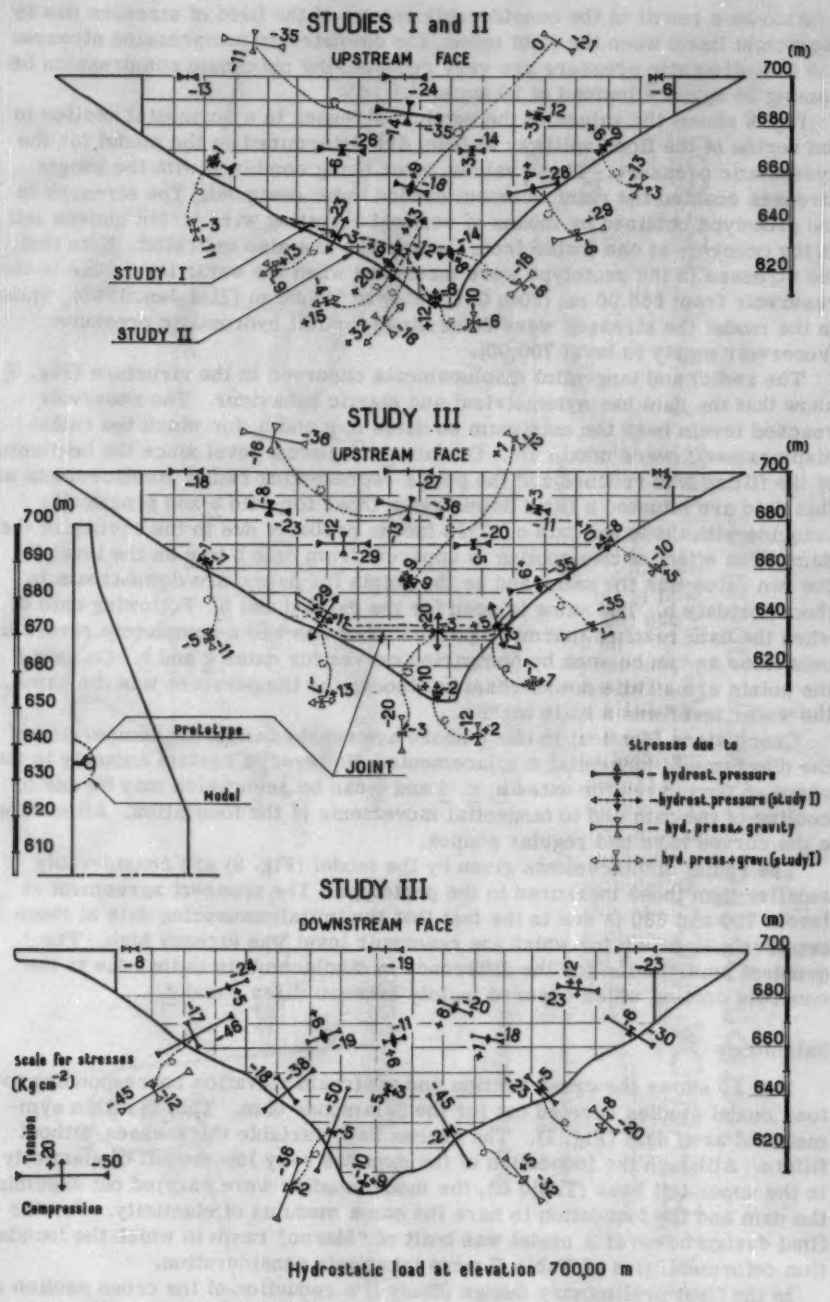


FIG. 7 –VENDA NOVA DAM. Stresses from model studies.

Also as a result of the considerable change of the field of stresses due to the weight itself when the joint opens, the downstream compression stresses due to hydrostatic pressure are very reduced, the maximum compression becoming 36 $kgcm^{-2}$ instead of 50 $kgcm^{-2}$.

Fig. 8 shows the values of the vertical stresses in a horizontal section in the region of the flood spillway at level 690 determined on the model for the hydrostatic pressure. These values, after being combined with the weight stresses enabled the reinforcement needed to be designed. The stresses in the prototype obtained by means of vertical vibrating wire strain meters left in the concrete at one meter from the surface are also indicated. Note that the stresses in the prototype were measured when the water level rose in the reservoir from 668.00 m (30th Oct. 1954) to 700.00 m (31st Jan. 1955), whilst in the model the stresses were determined for full hydrostatic pressure (reservoir empty to level 700,00).

The radial and tangential displacements observed in the structure (Fig. 9) show that the dam has symmetrical and elastic behaviour. The reservoir reached levels near the maximum on dates d, g and h, for which the radial displacements were maximum. On date e the lowest level since the beginning of the filling was reached and the points representing radial displacements at this date are situated a little downstream those for date b and practically coincide with those for date c. This fact is probably due to the cooling of the dam. The effect of this cooling is apparent from date b to c as the level for the two dates was the same and as the points for date c are downstream to those for date b. The same is seen for the dates d and h. Following date e, when the dam reached thermic equilibrium, it has had a completely reversible behaviour as can be seen by comparing curves for dates g and h. On date h the points are a little downstream as, though the temperature was the same, the water level was a little higher.

Conclusions identical to those above are reached from the comparison of the diagrams of tangential displacements. However, a certain anomaly in the shape of the curves for dates b, c, d and e can be seen which may be due to cooling of the dam and to tangential movements of the foundation. After date e the curves have had regular shapes.

The radial displacements given by the model (Fig. 9) are considerably smaller than those measured in the prototype. The apparent agreement at levels 700 and 680 is due to the fact that the initial measuring date at these two levels is date b for which the reservoir level was already high. The greatest contribution for the difference in displacements is that due to the concrete cooling which occured mainly between dates a and d.

Salamonde

Fig. 10 shows the cross section and upstream elevation corresponding to four model studies carried out for the Salamonde dam. This is a thin symmetrical arch dam (Fig. 1). The arches have variable thicknesses without fillets. Although the foundation of the dam has very low moduli of elasticity in the upper left bank (Table II), the model studies were carried out assuming the dam and the foundation to have the same modulus of elasticity. For the final design however a model was built of "Marco" resin in which the foundation deformabilities of Table II were taken into consideration.

In the first preliminary design (Study I) a reduction of the cross section at the heel and the construction of a socket to support the dam had been

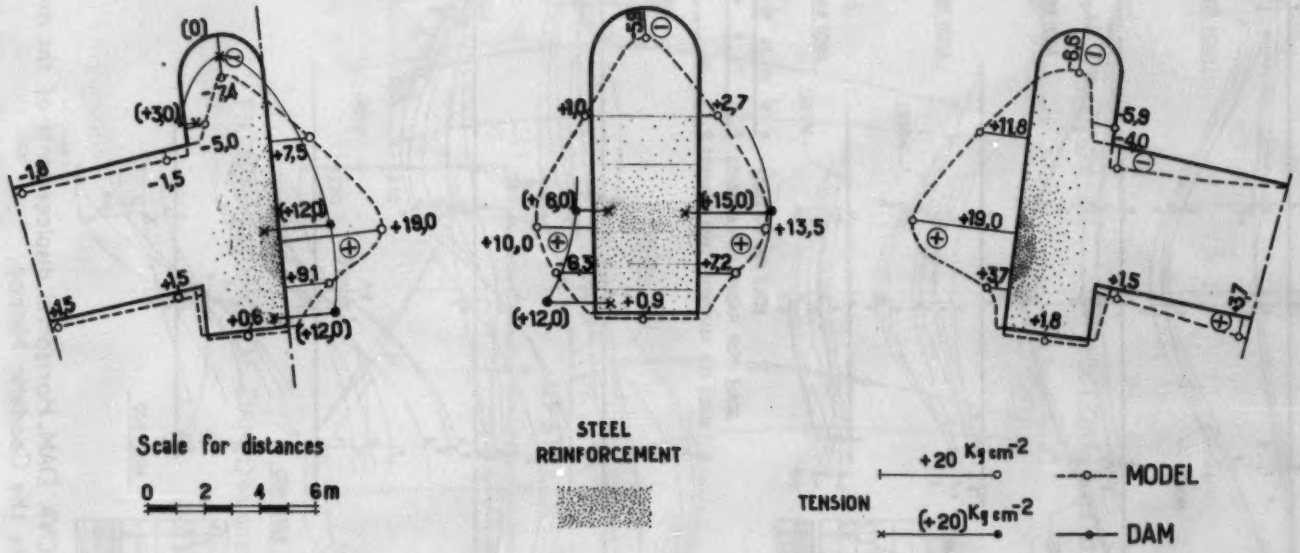


FIG. 8 –VENDA NOVA DAM. Vertical stresses in a horizontal section of the spillway openings.

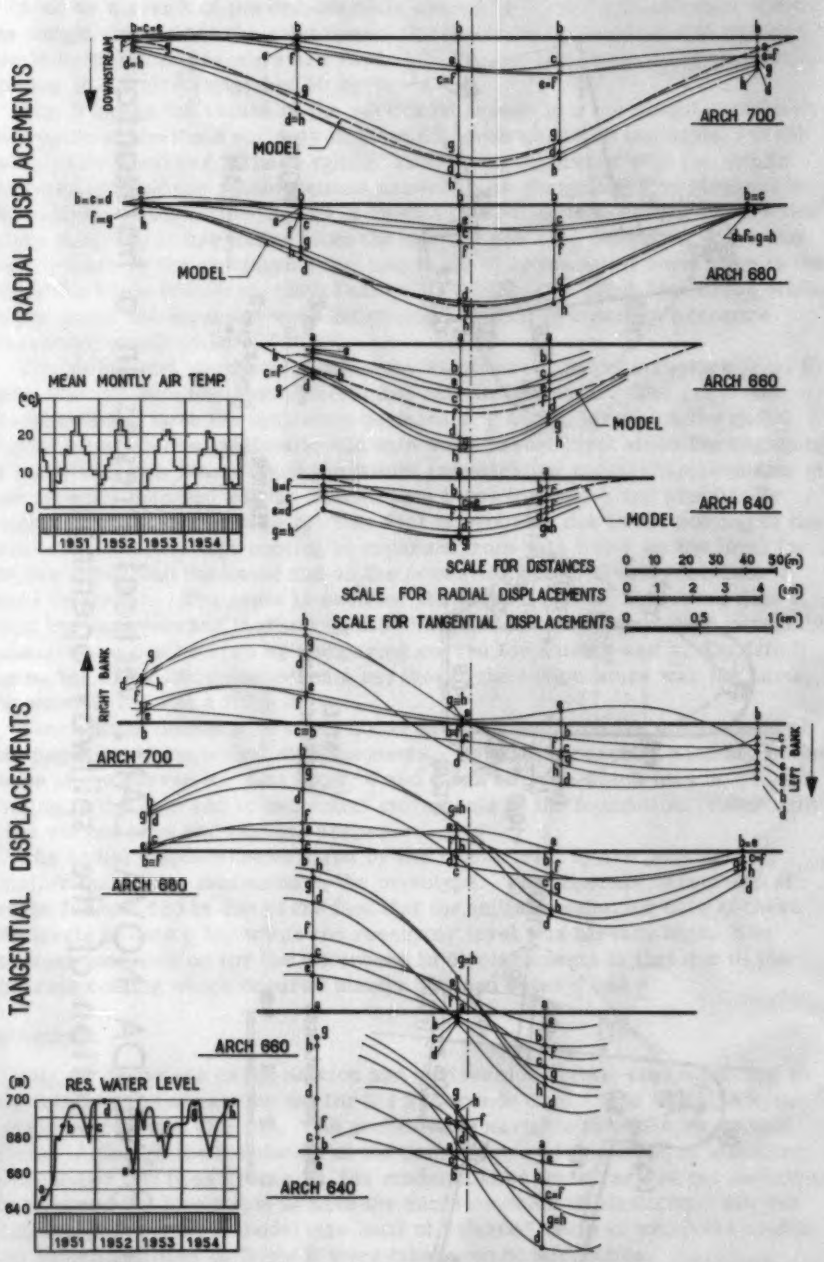


FIG. 9 – VENDA NOVA DAM. Horizontal displacements of the arches observed by the Geodetic Method.

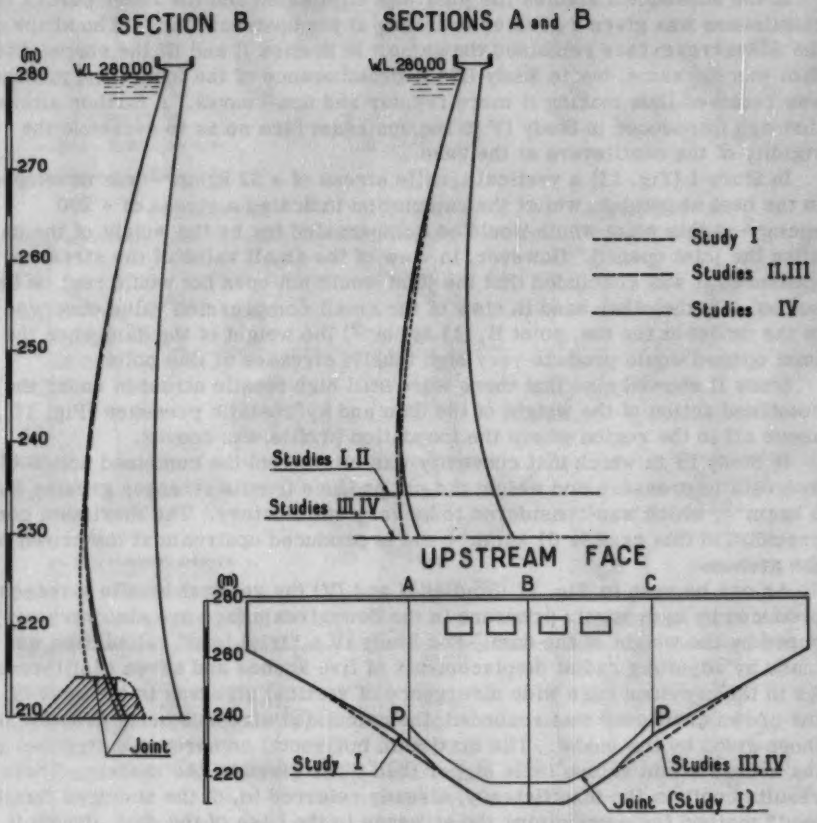


FIG. 10 – SALAMONDE DAM. Designs tested.

foreseen (Fig. 10). The joint between the socket and the dam would be closed by copper sheets and a gallery would be left inside for inspecting the joint. The idea was to give great flexibility to the base of the crown cantilever as it was supposed that the tensile stresses which would develop at the heel would be neutralised by the compression due to the weight of the dam, which would increase very much when the joint opened. This conception was based on a simplified calculation with radial adjustments of the crown cantilever and seven arches.

In the subsequent studies the joint was eliminated and the lower part of the cantilevers was given a greater overhang at the upstream face. The shape of the downstream face remained the same. In Studies II and III the shape of the dam was the same, but in Study III the protuberance of the foundation profile was removed thus making it more regular and non-convex. A further alteration was introduced in Study IV to the upstream face so as to decrease the rigidity of the cantilevers at the base.

In Study I (Fig. 11) a vertical tensile stress of + 32 $kgcm^{-2}$ was developed in the heel at point A, whilst the calculation indicated a stress of + 200 $kgcm^{-2}$ at this point which would be compensated for by the weight of the dam when the joint opened. However, in view of the small value of the stresses measured it was concluded that the joint would not open but would rest on the socket. On the other hand in view of the small compression value observed in the model at the toe, point B, (11 $kgcm^{-2}$) the weight of the dam when the joint opened would produce very high tensile stresses at this point.

Study II showed also that there were still high tensile stresses under the combined action of the weight of the dam and hydrostatic pressure (Fig. 11), above all in the region where the foundation profile was convex.

In Study IV in which that convexity was eliminated the combined action of hydrostatic pressure and weight did not produce tensile stresses greater than 6 $kgcm^{-2}$, which was considered to be very satisfactory. The maximum compression in this case is 61 $kgcm^{-2}$ and is produced upstream at the crown of the arches.

As can be seen in Fig. 11 (Studies II and IV) the vertical tensile stresses produced by hydrostatic pressure in the downstream face are also very reduced by the weight of the dam. For Study IV a "trial load" calculation was made by adjusting radial displacements of five arches and seven cantilevers. As in the previous case wide divergence of vertical stresses in the base of the crown cantilever was recorded, the calculated stresses being greater than those given by the model. The maximum horizontal compressive stresses in the arches attain values little higher than those given by the models. These results confirm the insufficiency, already referred to, of the abridged "trial load" method for determining the stresses in the base of the dam, though it gives approximate values of the compressive stresses in the arches.

In this case, too, the considerable influence of the convexity of the foundation surface on the behaviour of the arch dams was verified. This study also showed how small alterations of thickness, easily reproducible in the models, can make an important contribution to the reduction of tensile stresses in the base.

In Fig. 12 the curves are plotted of radial and tangential displacements of five cantilevers of Salamonde dam obtained by the goedetic method on five different dates of the years 1953, 1954 and 1955. As this dam is very thin, it had already reached thermic equilibrium when the filling of the reservoir was begun. Thus the differences between the curves of displacements

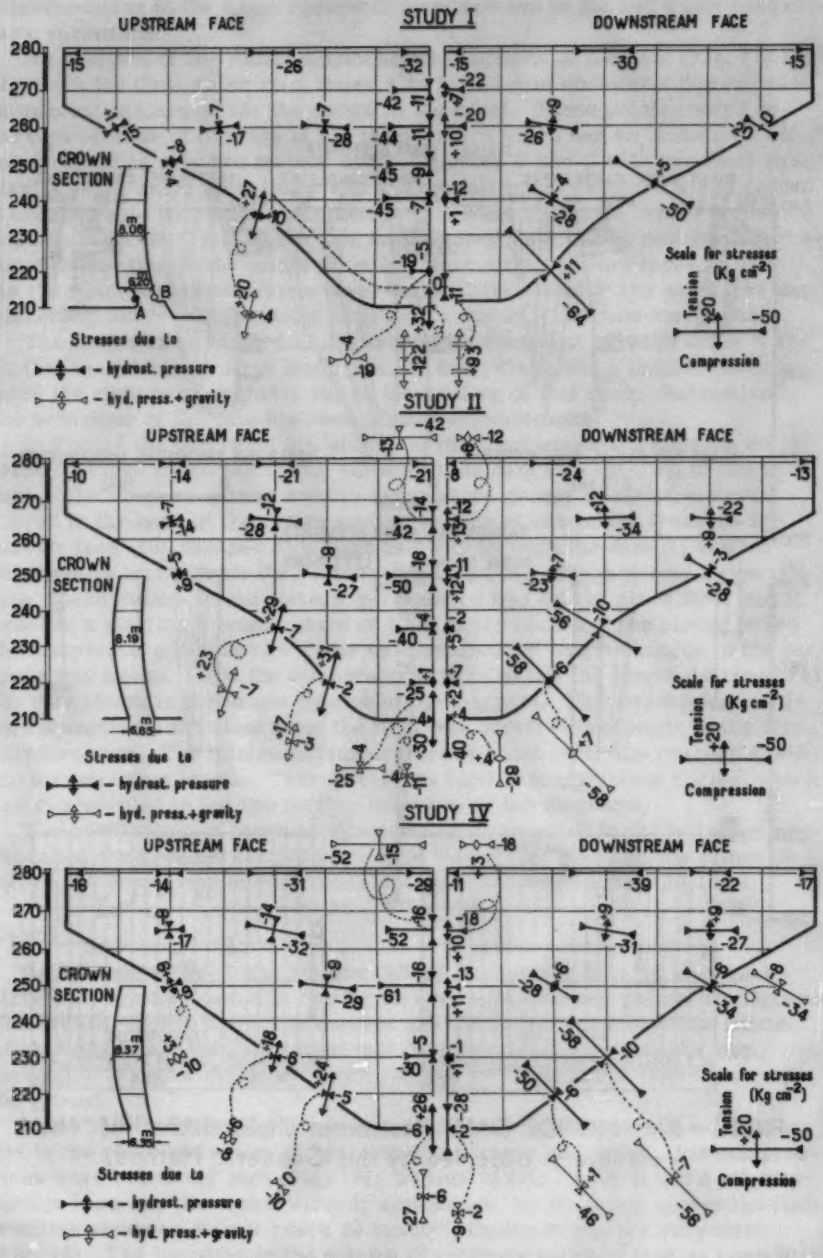


FIG.11 –SALAMONDE DAM. Stresses from model studies.

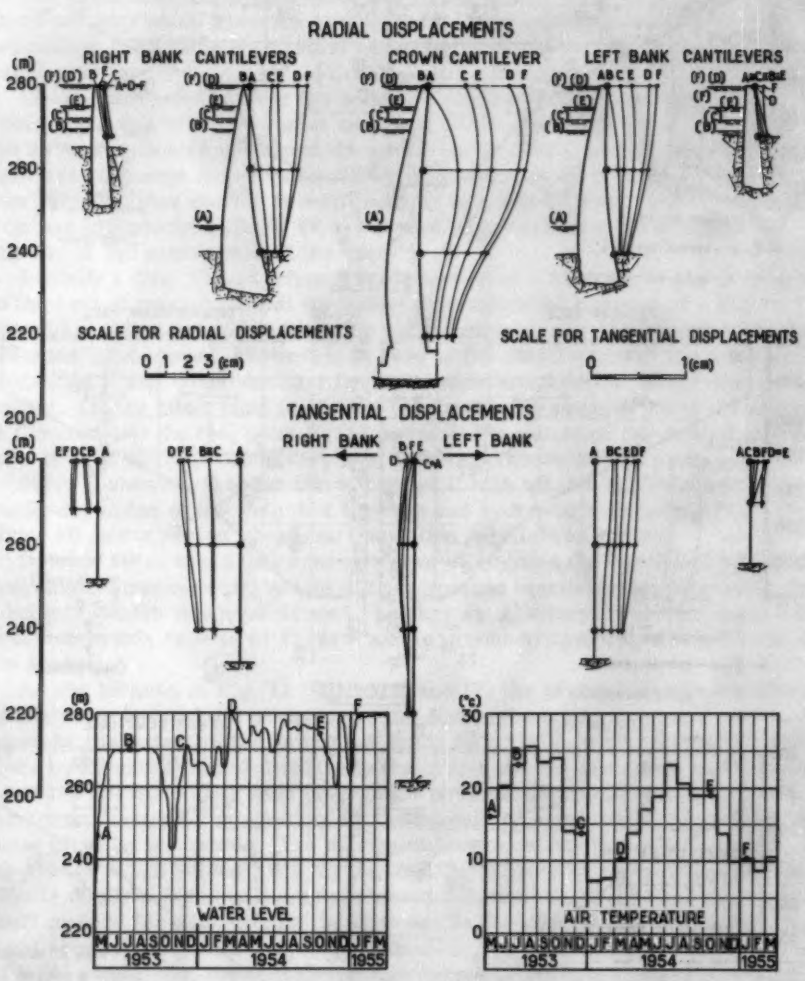


FIG.12 – SALAMONDE DAM. Horizontal displacements of the cantilevers observed by the Geodetic Method.

corresponding to the same reservoir level are due to air and water temperature variations.

An analysis of the radial displacement diagrams of the dam (Fig. 12) shows that with the first water rise, dates A to B, the dam underwent downstream displacements except for the points in the crest. These points moved upstream because of the rise of air temperature which has an immediate effect, as the crest is only two meters wide. On dates B and C with the water practically at the same level there was a large fall in temperature which caused a considerable downstream displacement. When the water level reached its maximum (dates D and F) the dam experienced its greatest downstream displacements, though the points of date F were slightly below those of date D. As the concrete temperatures observed on dates D and F are equal, the displacement between these dates is probably due to foundation movement.

The diagrams of tangential displacements show that between dates A and B, that is, during the first loading of the dam, there was a displacement toward the right bank probably due to the settling of this bank. Subsequently the behaviour of the dam has been almost symmetrical.

In Fig. 13 the diagrams are plotted of the concrete length changes registered by three no-stress strain meters in the dam as a function of temperature. The three no-stress meters serve three groups of strain meters placed in the base of the crown cantilever, one at one meter from the upstream face (1u), another at a point half way through the dam (1) and the third at one meter from the downstream face (1d). The concrete in the region where the no-stress meters were placed was laid at about 20°C and it reached a maximum temperature of 43°C at the centre of the block. When the temperature reached 36°C the measurement of length changes in the concrete was begun. Once the temperature began to fall the length decreased at the rate shown in the longer branch of the diagram. This branch has a different angular coefficient from the first one, which corresponds to the temperature rise. The minimum temperature of about 8° C was reached and so too the minimum length. There were two further temperature cycles, which are represented in the two further branches of the diagrams.

The coefficients of thermal expansion corresponding to the branches representing the greatest temperature falls were calculated and the values obtained are very close to each other and quite acceptable (Fig. 13).

Cabril

Cabril dam (Figs. 1 and 14), the tallest Portuguese dam, is a thin arch structure, symmetrical and resting on very homogeneous granite foundations. The arches have variable thicknesses and the downstream face has fillets. After Study I, a foundation socket was designed which thickens the dam, makes the shape of the valley more regular and diminishes the stresses on the ground.

In the models of Study I it was found that although there were no tensions due to the combined action of hydrostatic pressure and weight, the compressions were relatively high (Fig. 15). In view of this, Study II had a slightly thicker base and the socket already mentioned. In the actual socket the compression stresses did not reach 30 $kgcm^{-2}$, though it was not very thick (Fig. 14). The increase in the volume of concrete between Studies I and II was only from 348,000 to 354,000 m^3.

In Study II the compressive stresses (Fig. 15) were only 53 $kgcm^{-2}$ and

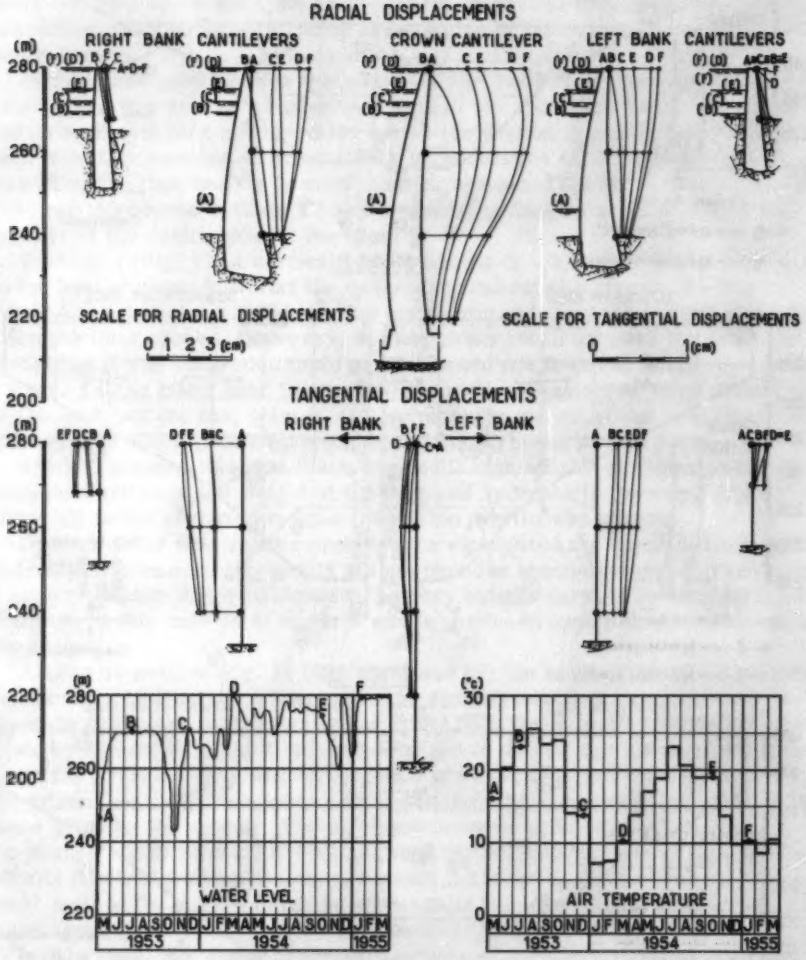


FIG.12 – SALAMONDE DAM. Horizontal displacements of the cantilevers observed by the Geodetic Method.

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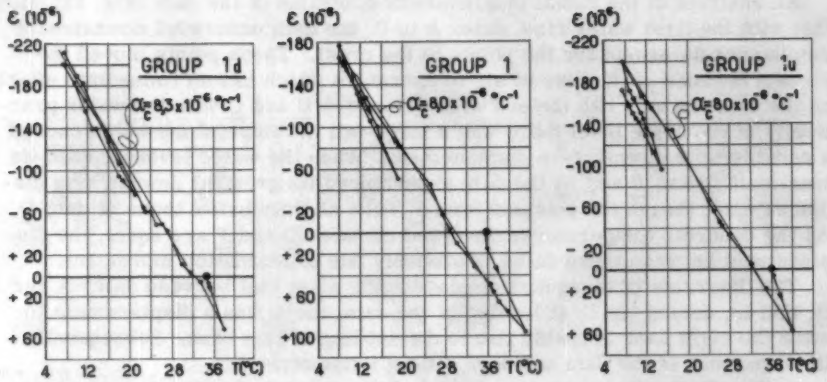

FIG. 13–SALAMONDE DAM. Strains versus temperatures measured by means of no-stress strain meters.

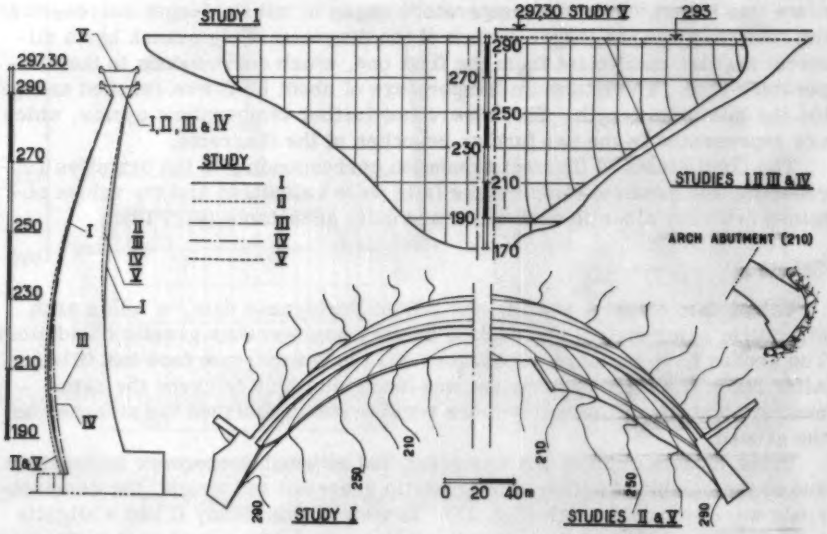

FIG. 14 –CABRIL DAM. Studies tested.

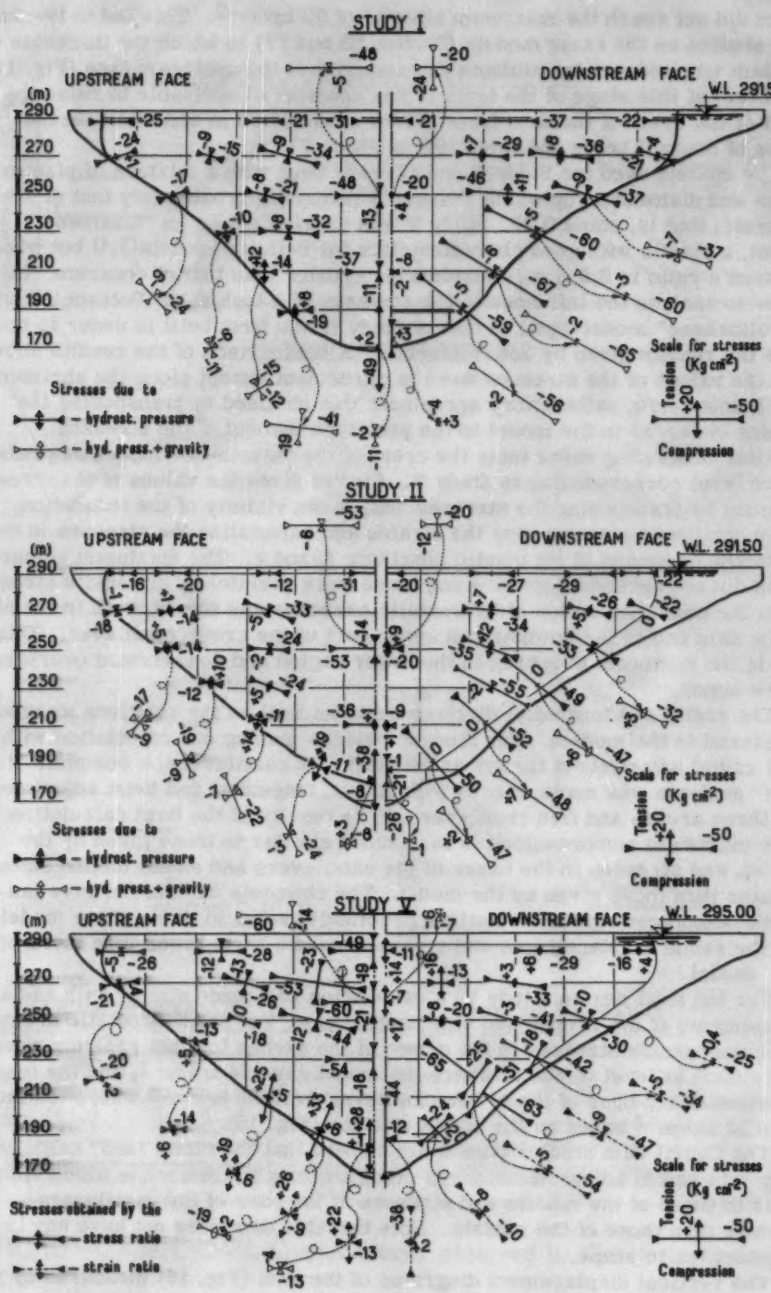

FIG.15– CABRIL DAM. Stresses from model studies.

hence did not reach the maximum allowed of 65 $kgcm^{-2}$. This led to two further studies on the same models (Studies III and IV) in which the thickness of the dam was reduced by cutting away material at the upstream face (Fig. 14). However, at this stage of the tests it was considered advisable to raise the level of the crest of the dam from 293.00 m to 297.30 m and maintain the shape of Study II below the level 290.m (Study V).

The models used for Study II and IV were built with a mixture of plaster of Paris and diatomite, for which Poisson's ratio is approximately that of the concrete, that is, about 0.20. Study V was carried out on an "alkathene" model, a plastic with good characteristics for building models[3,4] but whose Poisson's ratio is 0.5, i.e., considerably greater than that of concrete. In order to analyse the influence in the stresses of a high value Poisson's ratio, an "alkathene" model equal to that of Study II was first built in order to compare the results given by both materials. A comparison of the results showed that the values of the stresses were in agreement except along the abutment. For points there, satisfactory agreement was obtained by transposing the strains observed in the model to the prototype instead of the stresses.

After concluding these tests the crest of the "alkathene" model was raised to the level corresponding to Study V. Fig. 15 gives the values of the stresses obtained by transposing the stresses and, in the vicinity of the foundation, those obtained by transposing the strains and calculating the stresses in the prototype by means of its elastic constants E and ν. The maximum compressions did not exceed 65 $kgcm^{-2}$ and there were practically no tensile stresses from the combined action of hydrostatic pressure and weight, both in the heel of the dam and in the downstream upper part of the crown cantilever. This is due to the upstream overhang in the lower region and downstream overhang in the upper.

The radial and tangential displacements as well as the rotations were also measured in the models. For Study I, besides making one calculation with only radial adjustments for six arches and nine cantilevers, a complete "trial load" analysis was made, that is with radial, tangential and twist adjustments, for three arches and five cantilevers. The results of the first calculation gave maximum compressions in the arches similar to those given by the model, and stresses in the bases of the cantilevers and radial displacements greater than those given by the model. The complete calculation gave tangential displacements and rotations practically equal to those of the model[4] but the radial displacements and stresses were a little lower than those of the model.

For the final design (Study V) a calculation was made simply with radial adjustments of six arches and nine cantilevers. For the hydrostatic pressure the compressive stresses in the crown of the arches for this analysis were 60 $kgcm^{-2}$ at level of 230 m where the model gave 54 $kgcm^{-2}$, and the tensile stresses at the base of the crown cantilever were 30 $kgcm^{-2}$ where the model gave 22 $kgcm^{-2}$ based on the strain relation (Fig. 15).

The Cabril dam studies once again showed that the "trial load" calculation, only with radial adjustments, gives compressions in the arches which approximate to those of the models and stresses at the base of the cantilevers greater than those of the models. Note that this dam does not have any irregularities in shape.

The vertical displacement diagrams of the dam (Fig. 16) measured by precision levelling show that, during the first loading, the crest rose (dates A to C). This rise is in good agreement with that found in the model; between

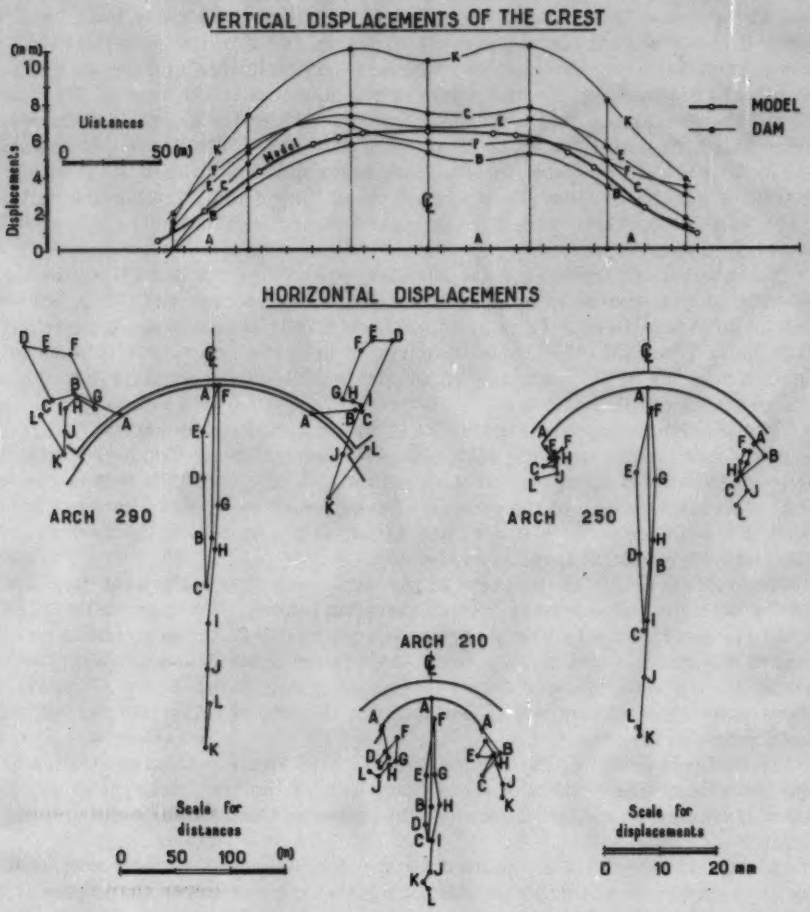

FIG. 16 – CABRIL DAM. Displacements observed by Geodetic Methods.

those two dates the effect of temperature variations was not apparent, as the lower 100 meters of the dam had a mean temperature drop of 1°C and the upper 30 meters a mean rise of 4°C (Fig. 17). The drop of water level between dates C and F did not cause the crest to return to its initial position even when taking into consideration the temperature variations and the rise of the abutments revealed by the end points of the diagrams. The rise of the water level between dates F and K caused a further rise of the crest. As however there was a mean temperature drop in the dam of 5°C the curve corresponding to date K should not be far from that corresponding to date F. It was therefore concluded from the observations of the vertical displacements that there must be another cause for the rise of the crest besides the hydrostatic pressure and temperature.

The analysis of the horizontal displacements (Figs. 16 and 17) shows that the dam has an almost reversible and symmetrical behaviour. Note, however, that from date A to date F, in spite of there having been a large temperature rise above level 230, the points observed in the crown cantilever did not go upstream of the positions occupied on date A. This is probably due to a displacement of the foundation.

The maximum reservoir water level was attained on the second filling (dates K and L), the greatest displacements being observed on date K at points at level 290 and 250 and on date L at points at level 210. This fact is due to the mean temperature of the concrete having risen by 10°C in the arch at level 290 between dates K and L, and having fallen by 1°C in the arch at level 250 and 2°C in that at level 210 (Fig. 17).

The lateral points of the crest of the dam (arch 290) underwent displacements strongly influenced by temperature variations. Between dates C and D, when the mean temperature of the crest rose by 10°C, these points were displaced upstream by the same amount as between dates A and C. With the fall of the mean temperature of the crest following date D till date K (Fig. 17) these points moved continually downstream in spite of important variations in hydrostatic pressure.

The lateral points of the arches 250 and 210, whose paths are similar to the paths of displacements observed on points of the rock near the dam,(6) show movements similar to those of the crown of the arches at the various levels.

In Fig. 17 diagrams are plotted for the displacements of the crown cantilever observed by the geodetic method against the reservoir water level. The curves of the mean temperatures of the concrete and the mean monthly air temperature also, against the reservoir level, are drawn in the same figure. The displacement diagrams, when taking the temperatures into consideration, reveal the general reversibility of the dam. Besides the irreversibility between dates A and F already mentioned, these diagrams also show an anomaly in the behaviour during the second filling. In fact on date H the points of the crown of the arches 250 and 210 were situated upstream to the positions occupied in date D, even though for these two dates the reservoir levels were equal and the concrete temperatures were lower in date H than in date D. This is probably explained by the final grouting of the joints which was carried out on date F, when they opened in the lower part.

Fig. 18 shows the diagrams of the temperatures measured in four cross sections of Cabril Dam at 1 meter from the two faces and half way through, from 1952 till 1955. In the same figure are traced the diagrams of the calculated temperatures for the intermediate points which agree very closely with the temperatures observed at these points.

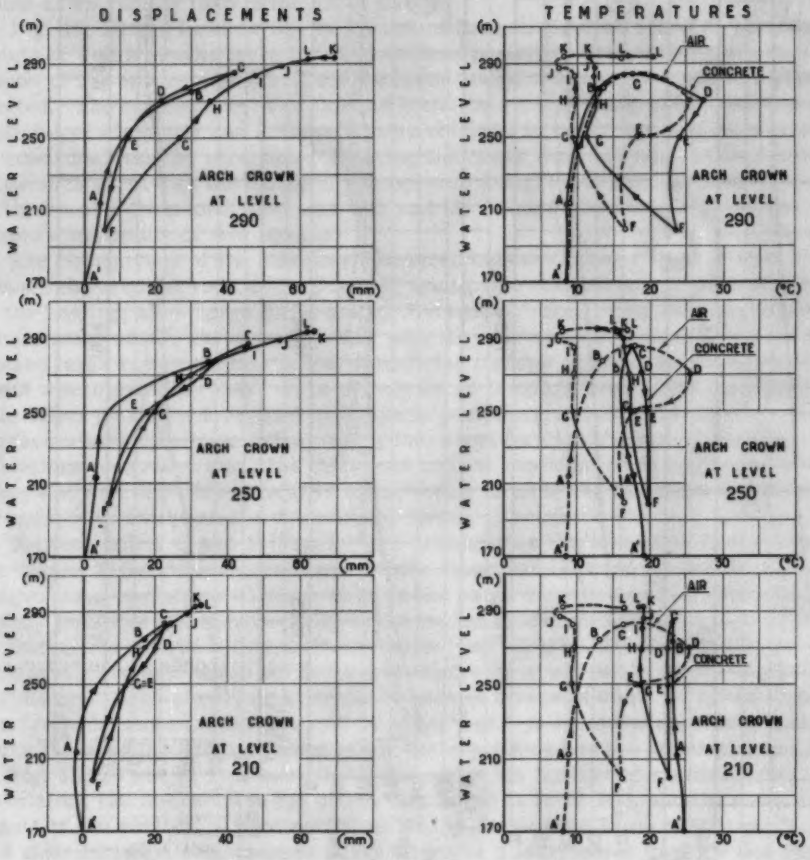

FIG.17 - CABRIL DAM. Observed radial displacements and mean concrete temperatures of the crown cantilever.

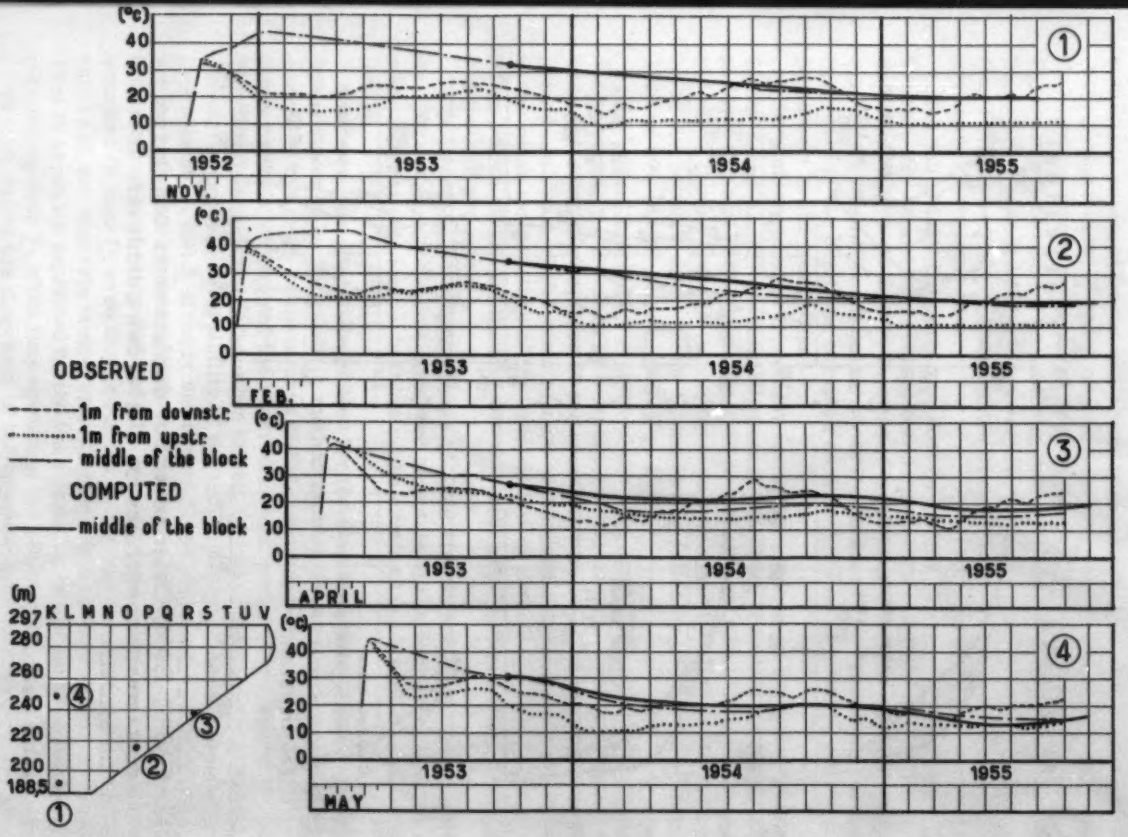

FIG. 18 -CABRIL DAM. Observed and computed temperatures.

In the lower part of the dam a low heat type of cement was used which developes the heat very slowly. The concrete of the dam had 250 kg of cement per cubic meter and 1.5 m lifts were poured at intervals of 4 days. As diagrams 1 and 2 show, the maximum temperature was only reached three months after laying. The upper half of the dam was built with normal Portland type cement and also with 250 kg per m^3. The maximum temperature was reached three days after laying and the heat of hydration was dissipated much more rapidly than in the lower part.

Fig. 19, 20 and 21 show the variations of the principal stresses at various points at 1 m from the faces between various periods. The temperature variations of the concrete at 1 m from the faces and at the mean surface are also plotted. The stresses were calculated from the strains using values for the modulus of elasticity and Poisson's ratio obtained in rapid tests on specimens of concrete from the structure. Ignoring the creep does not lead, in this case, to serious errors as the filling of the reservoir began when the greater part of the concrete was over one year old, and the loading and unloading cycles lasted a maximum of five months.

The comparison of the stresses measured between dates A' and C with those measured between dates C and F, taking into consideration the reversal of the loading, show large differences. Between A' and C (Fig. 19) the fall in temperature inside the dam, together with the rise in temperature of the faces caused large compressions in the superficial regions of the dam, except at the crest where probably there were not any large temperature gradients. This skin effect probably developed hydrostatic compressions at 1 m from the surface whose maximum value, taking into consideration the temperature variations measured along the thickness and the modulus of elasticity of the concrete, was probably around 15 $kgcm^{-2}$ at 1 m from the upstream face and 30 $kgcm^{-2}$ at 1 m from the downstream face.

Between dates C and F (Fig. 20) the decompressions are much lower than the values of the compressions previously measured between A' and C, as the temperature variations continue to be in the same direction and of approximately the same value, especially in the region below arch 260.

During the second filling, between dates F and K (Fig. 21), large falls in temperature took place in the faces and relatively small ones inside. These variations caused maximum hydrostatic tensile stress of about 30 $kgcm^{-2}$ at 1 m from the downstream face and 10 $kgcm^{-2}$ at 1 m from the upstream face which relieved the compressions at the faces produced by the water pressure.

Fig. 19, 20 and 21 also have the diagrams of the horizontal stress variations along the thickness in the crown cantilever at level 250, and the diagrams of the vertical stress variations at the base of the same cantilever. The distribution of the stresses is far from the linear, which must be due to the temperature variations.

It should be noted that the agreement between the directions of the principal stresses observed in the various dates and those given by the models shows that the skin effect produced by the temperature variations is a state of hydrostatic stress.

A comparison of the maximum stresses given by the models with those observed on the structure gives a very good agreement when the measured values of the elastic constants and the differences between the mean temperatures of the concrete (Fig. 17) and the temperature at the point are taken in full consideration. For example the maximum compressive stress (point 6u) given by the model, for the water at level 290, is 50 $kgcm^{-2}$, whilst on the

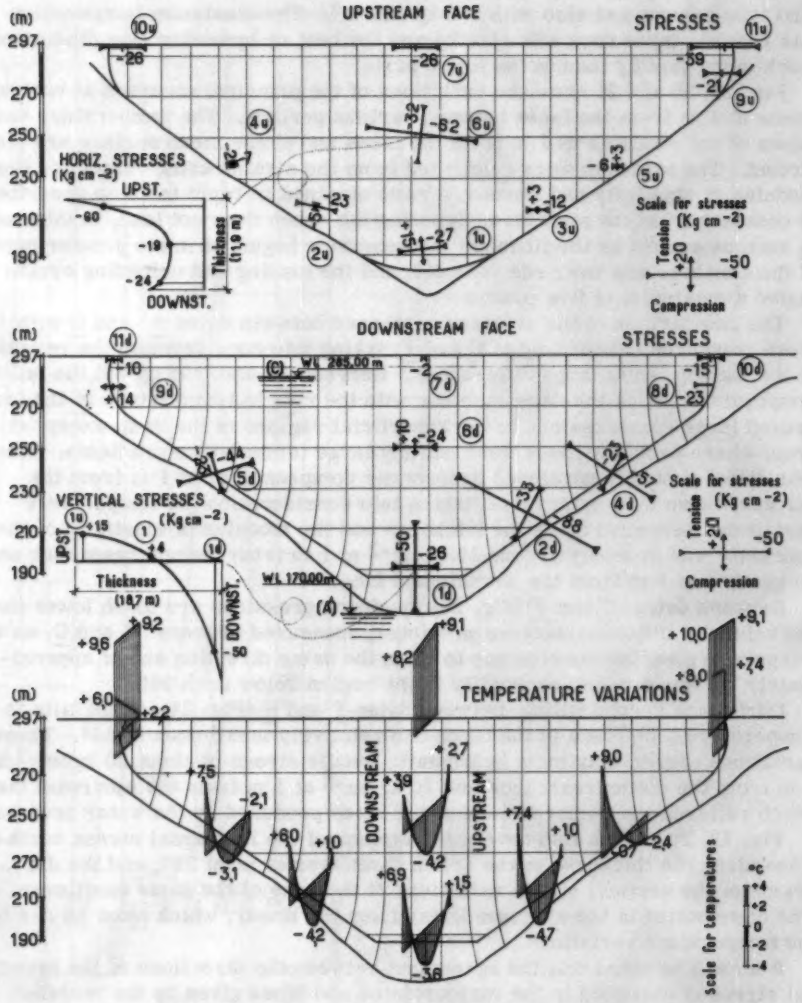

FIG. 19 – CABRIL DAM. Observed stresses and temperatures during the first loading (Date A' to C).

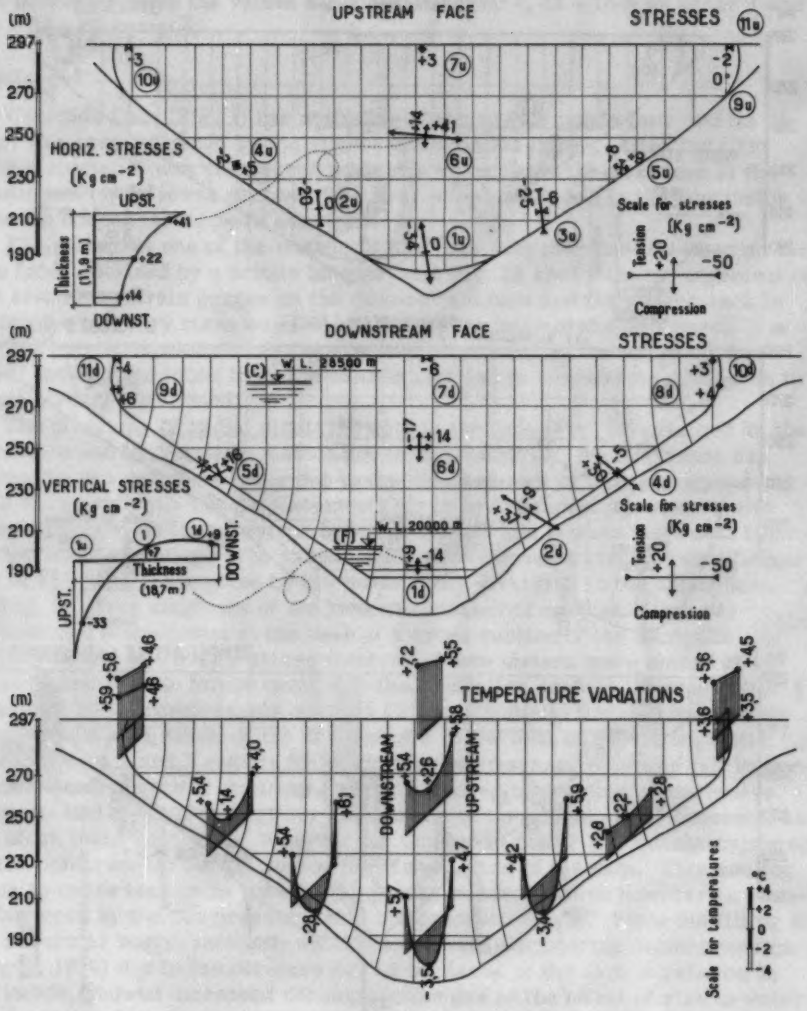

FIG. 20–CABRIL DAM. Observed stresses and temperatures during the first unloading (Date C to F).

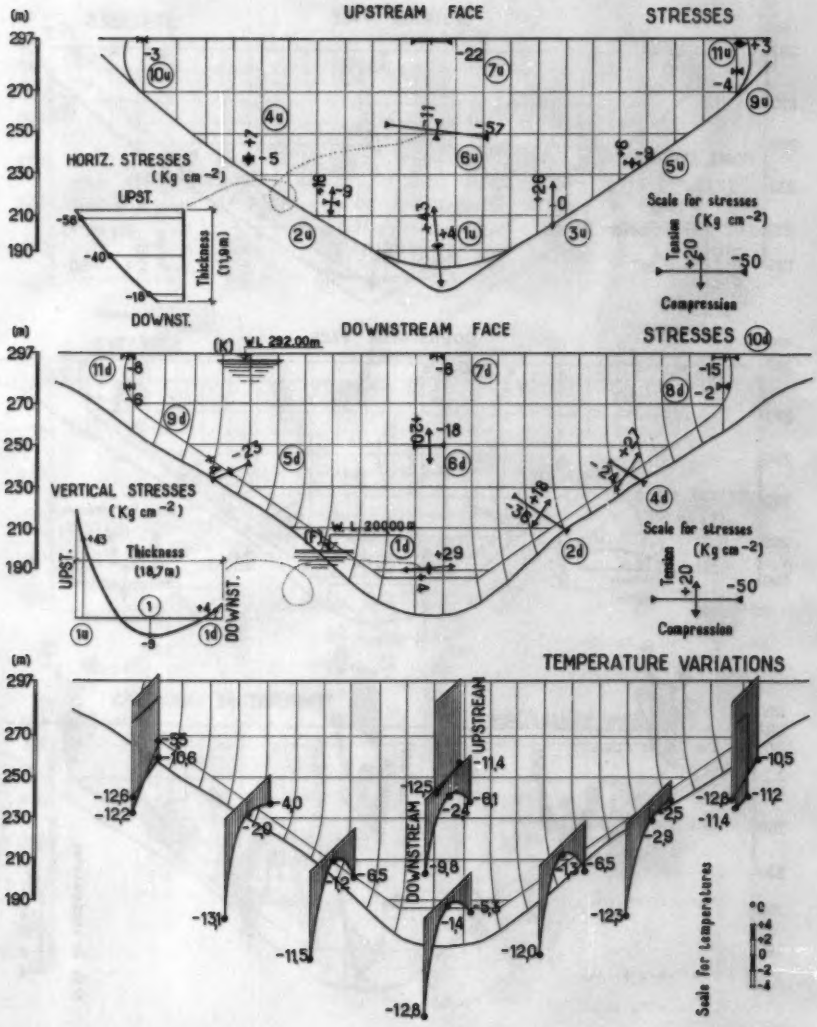

FIG. 21 – CABRIL DAM. Observed stresses and temperatures during the second loading (Date F to K).

prototype between dates A' and C the hydrostatic pressure produced a stress of 62 - 12 $kgcm^{-2}$ = 50 $kgcm^{-2}$; between dates C and F 41 + 9 = 50 $kgcm^{-2}$ and between dates F and K 57 - 6 = 51 $kgcm^{-2}$. The maximum downstream compression at level 220 (point 2d) given by the model is 60 $kgcm^{-2}$ whilst the prototype gives the values 88 - 14 = 64 $kgcm^{-2}$, 37 + 18 = 55 $kgcm^{-2}$ and 36 + 27 = 63 $kgcm^{-2}$.

Caniçada

Caniçada Dam (Fig. 1) has a similar shape to Salamonde Dam and its design was greatly helped by the studies made on the latter. After the first model studies it was considered advisable to increase the thickness at the abutments of the lower arches. The final model study led to the conclusion that the shapes tested were satisfactory.

Fig. 22 shows one of the models of this dam with the isostatic lines on the two faces obtained by a brittle lacquer, and Fig. 23 shows the arrangement of the electrical strain gauges on the downstream face and the rubber sack in which the mercury rises so as to reproduce the hydrostatic pressure.

The comparisons between the stresses calculated by the abridged "trial load" method and those from the models also led to conclusions analogous to those already mentioned.

The diagrams of radial displacements in the dam (Fig. 24) obtained by the pendulum and the geodetic method agree satisfactorily, the difference between the two diagrams being due to the displacement of the reference point P of the pendulum. The displacements given by the model agree well with those observed on the prototype between dates C and D when there was little temperature variation. The values of the displacements given by calculation agree well with those given by the model, only diverging in the upper part.

Fig. 25 gives diagrams of the stresses measured upstream, halfway through and downstream at the base of a cross section of the Caniçada dam by means of Carlson stress meters. These meters were placed at 45° so as to receive the forces normal to the foundation surface. Stress meter 2 shows an initial compression of about 13 $kgcm^{-2}$ due to the rise in concrete temperature as a result of the development of the heat of hydration, whilst stressmeters 1 and 3 registered slight tensile stresses. With the fall in temperature and growth of the block, meter 1 started to register compressive stresses and meter 3 to register tension due to the weight of the concrete, as the block leans upstream. However the tension in meter 3 is counterbalanced by the compression due to the cooling of the inside of the dam. This cooling tends to cause tension as detected by stress meter 2, which however is counterbalanced by the compression from the concrete weight. When the filling of the reservoir began, meter 1, which was already registering decompression (August 1954) due to temperature fall on the faces of the dam in relation to the inside, showed increased decompression due to the effect of rise in water level. Meter 3 shows compression due to this rise in water level very clearly and meter 2 has intermediary values. At the beginning of May 1955 the temperature of the concrete at the faces began to rise and immediately the compression at meters 1 and 3 did so too, whilst meter 2 registered decompression for the same reason. With decrease in water pressure, which took place in October 1955, meter 3 showed decompression. This effect was accentuated by a fall in temperature of the downstream face which occured at the same time. Meter 1 registered an increase in compression due to fall in

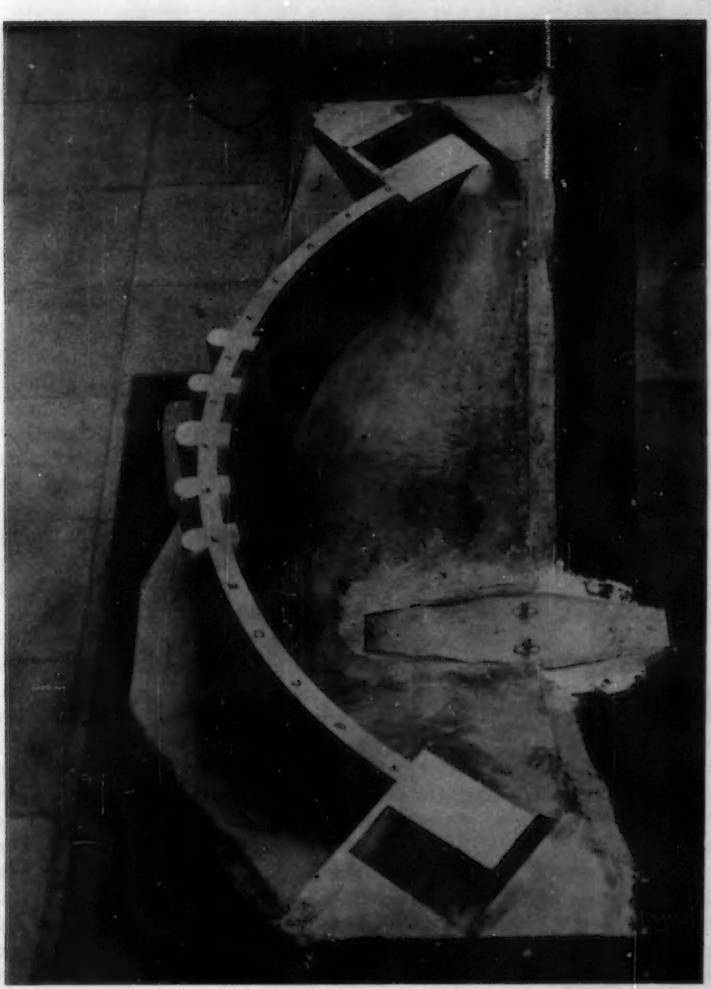

FIG.22-CANIÇADA DAM. Isostatic lines at the faces of the model

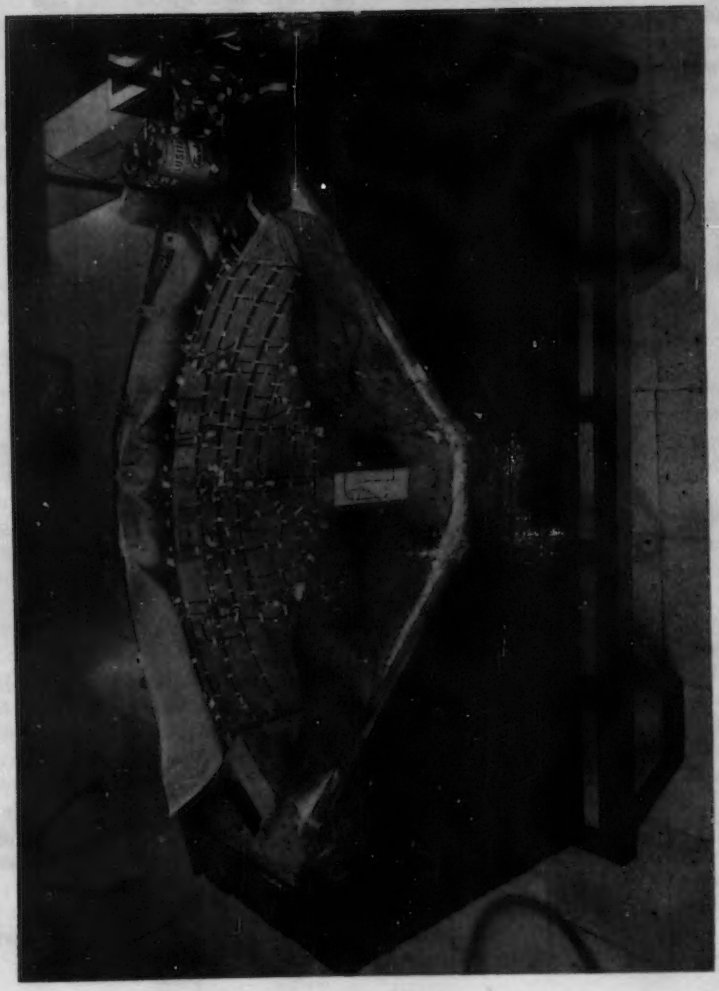

FIG.23-CANÇADA DAM. Strain gauges on the downstream face

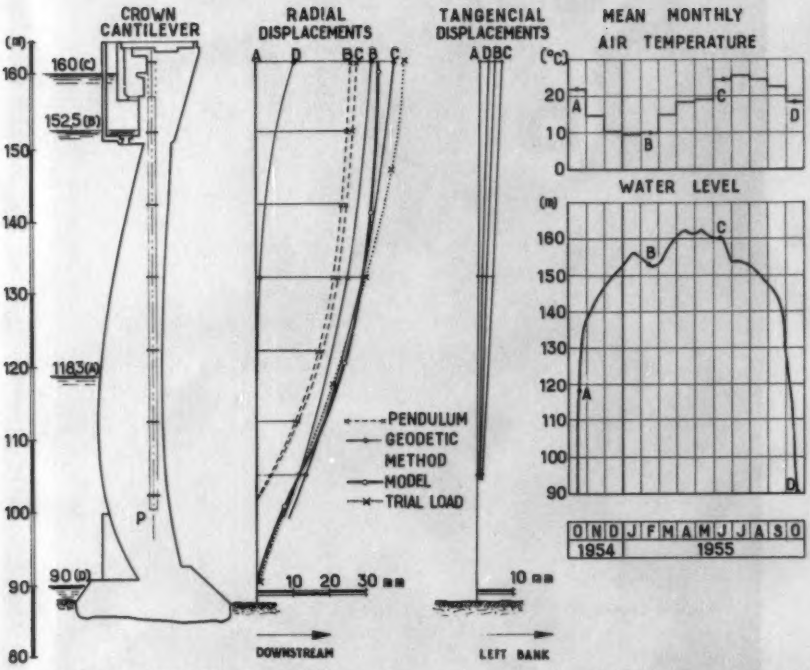

FIG. 24 - CANIÇADA DAM. Observed horizontal displacements of the crown cantilever.

water level. When the water rose again, meter 3 registered further compression and meter 1 decompression. Meter 2 also registered compression which was mainly due to the cooling of the faces rather than to hydrostatic pressure.

The model tests for this dam gave - 50 $kgcm^{-2}$ and + 18 $kgcm^{-2}$, for downstream and upstream respectively, at the same points and direction as the stress meters. Such stresses are much greater than those given by the stress meters, which were - 17 $kgcm^{-2}$ and + 10 $kgcm^{-2}$, between October 1954 and April 1955 when the reservoir was filled. The very much lower values given by the stress-meters can, in part but not completely, be explained by local thermal effects.

Before beginning the construction of the dam a study was made to estimate temperatures as it was wanted to accelerate the construction by laying 2 m lifts at intervals of four days. The proportion of cement used was 250 kg per m^3. This cement at the end of 3 days developed 63 calories per gram, at 7 days 71 and at 28 days 84 calories. Fig. 26 shows diagrams of recorded and calculated temperature for two cross sections of the dam which agree satisfactorily.

Bouçã

The design of Bouçã dam (Fig. 1), like the previous ones was studied in detail by models. It is a dam with accentuated double curvature and which can discharge a flow of about 2300 m^3 per sec. over the crest. The arches have variable thickness and have no fillets. There is a socket on the downstream side, as in Cabril Dam. Fig. 27 shows one of the models being worked.

The shape of the dam is so favourable that if the thickness was reduced to obtain a maximum stress of 65 $kgcm^{-2}$ it would be too thin for mass concrete construction.

Fig. 28 shows the isostatics for the two faces due to hydrostatic pressure, obtained by "stress-coat" and the values of the stresses due to this loading. For some of the most important points the values of the stresses due to combined effect of hydrostatic pressure and weight of the concrete are given.

The dam which is being observed for its first loading has not been grouted yet. Although a flood of 1500 m^3 sec^{-1} has flowed over the structure, when the upstream level reached 179.00 m, no anomaly has been observed in its behaviour.

Fig. 29 gives the stresses observed for the first filling at level 174. These stresses were obtained exactly as in the Cabril Dam. A certain asymmetry was recorded in its behaviour which was also verified in the displacements. This asymmetry appears to be caused by the different exposure to the sun of the two banks.

CONCLUSIONS

The experience obtained in Portugal leads to the conclusion that the thin arch dams, in relation to all other types, constitute the most economic and safe solution whenever the foundation characteristics permit it and the valley is not excessively wide. This last limitation is not, however, so common as is supposed. With regard to the foundations, too, the requirements are not

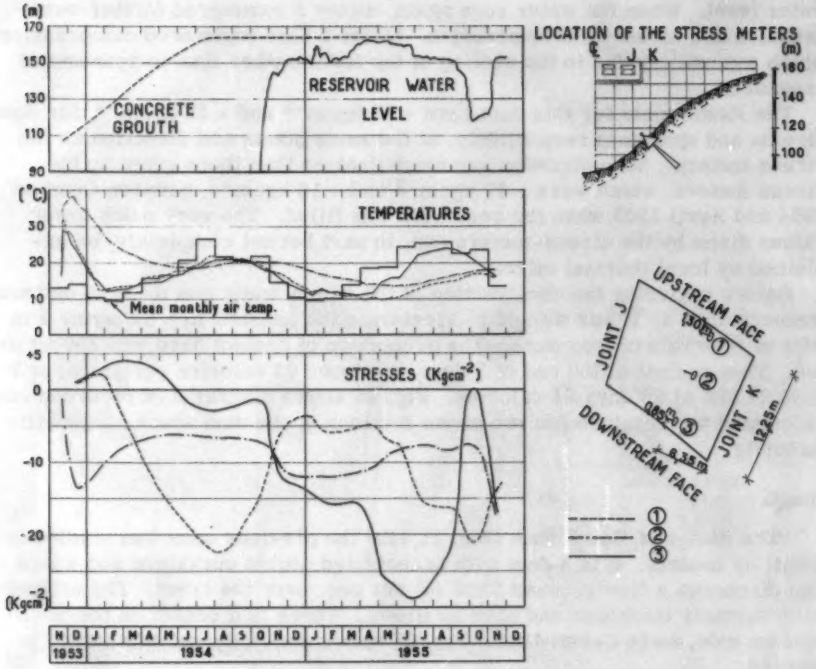

FIG. 25–CANIÇADA DAM. Stresses measured by means of Carlson stress meters.

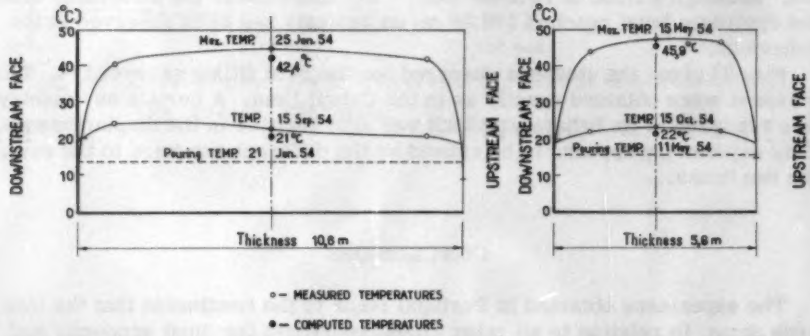

FIG. 26–CANIÇADA DAM. Measured and computed temperatures at two sections of the dam.

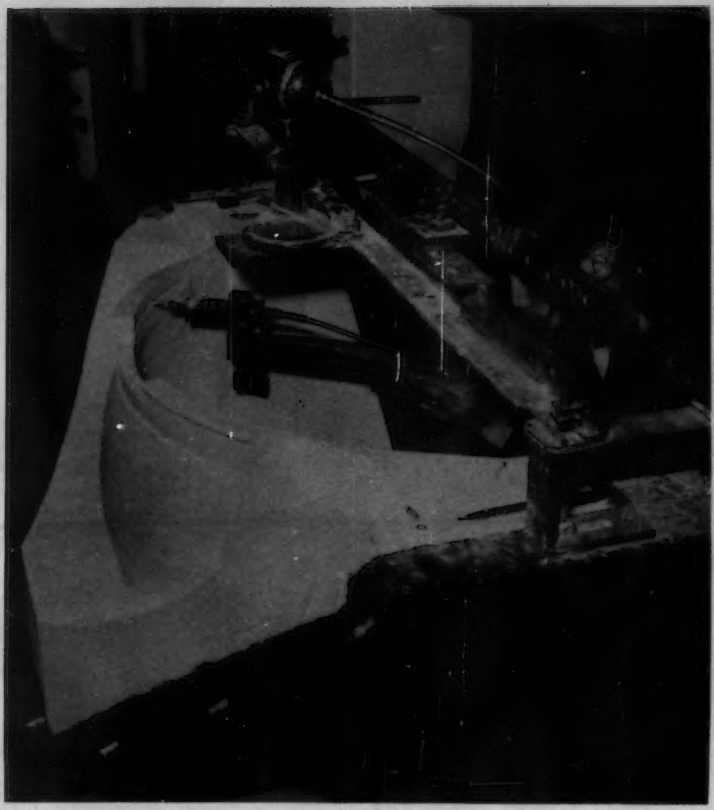

FIG.27 – BOUÇÃ DAM. Construction of the model

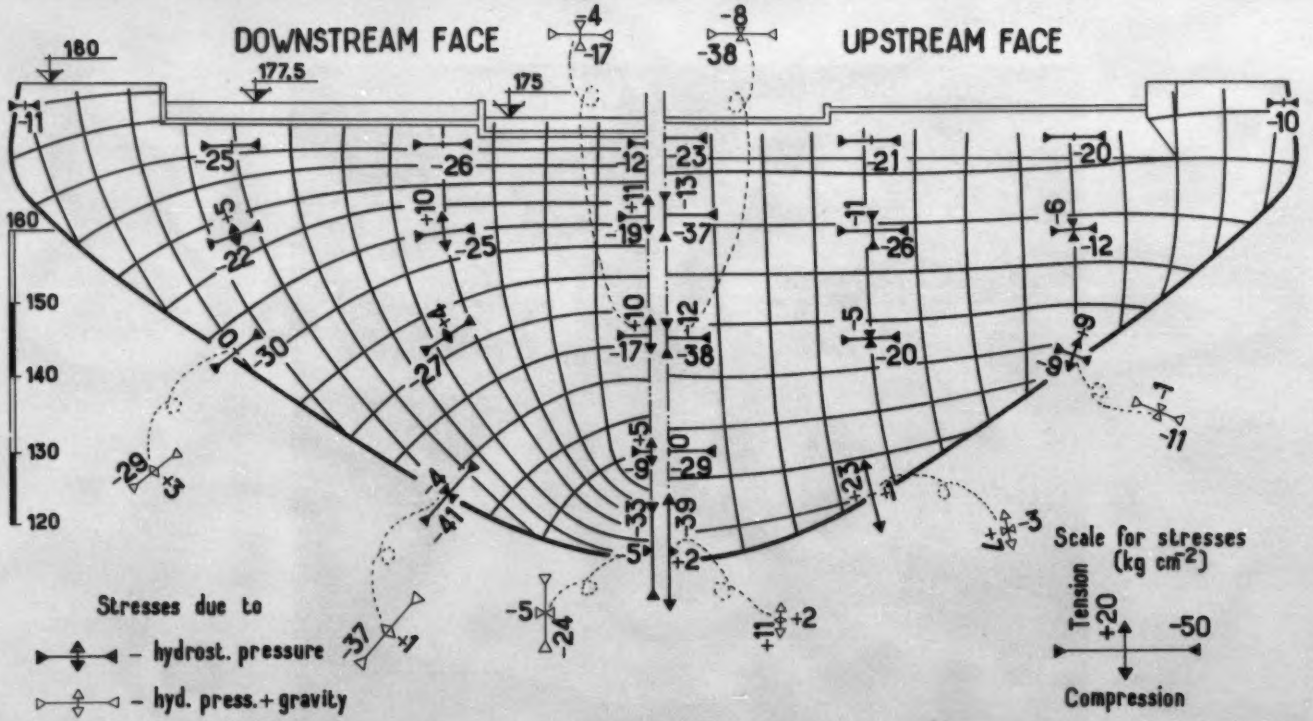

FIG. 28– BOUÇÃ DAM. Stresses from model studies (Mercury load at elev. 177,5).

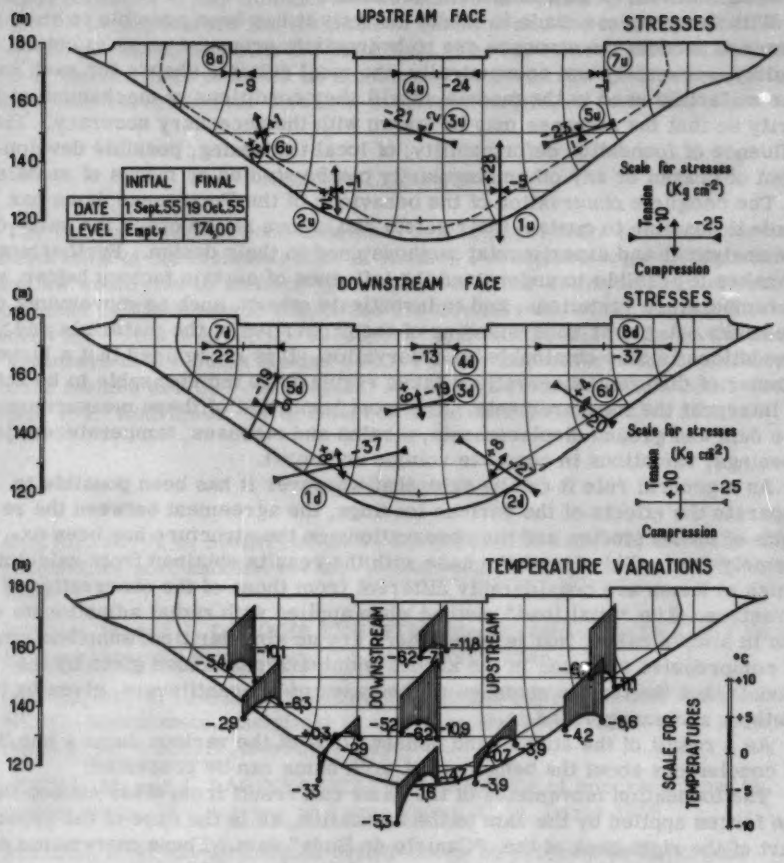

FIG. 29 – BOUÇÃ DAM. Observed stresses and temperatures during the first loading.

very exacting. In fact thin arch dams can be built on very deformable rocks. Only tests carried out on site over large volumes of rock can give information on the deformability of the foundations.

When the shapes, which depend on the site conditions, are carefully studied the arch dams can be very thin, veritable shells in fact. However, it is not advisable for the thickness to be less than certain limits both for reasons of construction and for ensuring the watertightness of the dam. In Portugal 2 m is being considered as a minimum thickness.

With the progress made in model analysis it has been possible to study by means of models the stresses due to hydrostatic pressure so as to obtain, rapidly, accurately and economically, the most suitable shapes for each case. The materials used in the models should obey conditions of mechanical similarity so that the stresses may be known with the necessary accuracy. The influence of foundation deformability, of local thickening, possible development of cracks or any other singularity can be studied by means of models.

The complete observation of the behaviour of the Portuguese dams has made it possible to control their safety and, above all, to check the value of the analytical and experimental methods used in their design. Furthermore it makes it possible to understand the influence of certain factors better, such as temperature variations, and to investigate others, such as movements of the banks. A better understanding of the properties of the materials and foundations is also obtainable by observation. It is recognised that a large number of different observations taken regularly is indispensable to be able to interpret the measurements. The most important of these measurements are dam and ground displacements, strains and stresses, temperatures, joint openings, variations in concrete volume and uplift.

As a general rule it can be said, that whenever it has been possible to separate the effects of the various loadings, the agreement between the results of model studies and the observations on the structure has been extremely good. This is not the case with the results obtained from calculations which at times are considerably different from those of the observation of the structure. The "trial load" method when applied with radial adjustments only, and in simple cases, that is, when there are no singularities, supplies values of compressive stresses in the arches which are near those given by the models, but the tensile stresses at the bases of the cantilevers, given by that method, are exaggerated.

As a result of the studies and observations of the various dams a number of conclusions about the behaviour of arch dams can be presented:

The foundation movements of the dams can result from other causes than the forces applied by the dam to the foundation, as is the case of the upper part of the right bank of the "Cantelo do Bode" dam. These movements can have very great effect on the behaviour of the structure.

Singularities in the foundations should be avoided. Normally convexities can increase the stresses both tensile and compressive. It is therefore always advisable to investigate the influence of singularities and to decide to what degree they can be permitted, by means of model studies.

Special attention should be paid to the stresses in the faces of openings in arch dams as important tensile stresses can develop there.

The thermal phenomena of the dams are complex but can be calculated. The dissipation of the heat of hydration causes considerable compression at the faces. If such compressions are not excessive they can contribute to the watertightness of the structure. However, once the heat dissipation causes

joint openings, the best means for obtaining suitable conditions for grouting in a short time is by artificial cooling of the concrete.

The displacements due to loss of heat and external temperature variations are very important. In the upper parts of the thinner dams they may reach values comparable to those due to hydrostatic pressure.

The external temperature variations produce a skin effect which consists of a hydrostatic state of stress parallel to the faces of the dam. The values of these stresses at any moment can be estimated by taking the difference between the temperature for the point in question and the mean temperature along the thickness on which the point lies. The agreement between the results of the stresses obtained on models and those on the actual dams, after making this correction for stresses of thermal origin, is remarkable.

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Discussion of "PRINCIPLES OF FEDERAL HYDRO-ELECTRIC POWER DEVELOPMENT"

by William Whipple, Jr.
(Proc. Paper 739)

GUS NORWOOD.[1]—The writer has 3 main points of disagreement with Colonel Whipple's paper—

1) Inclusion of "taxes foregone" when there is no basis in law for this.
2) Use of an interest rate higher than actual cost of money to the Federal government.
3) Use of shorter service life than that provided and recommended in Bulletin "F" of the Bureau of Internal Revenue which permits 150 years for FPC Account Number 322 on "Waterways, dams and reservoirs."

The following minor additional comments are submitted as supplementary to the above three major points of disagreement.

1) Page 739-1. The writer does not agree with the statement of generally accepted public policy in the 1920's. The Federal Water Power Act in section 7b and other provisions indicates when a project should be built by the United States itself. It became law June 10, 1920.
2) Page 739-2. Colonel Whipple overlooks the following statement in an Army report:

"In general, any partnership relation between the United States and a private corporation is necessarily to be closely scrutinized as the results in the past have been that the Government as party to such agreements has usually suffered thereby."

(From the report of the Rivers and Harbors Board of the Corps of Engineers dated January 22, 1909 and printed as House Document Number 14, 60th Congress, Second Session, which report recommended disapproval of the Muscle Shoals Hydroelectric Power Company proposal to construct dams on the Tennessee River provided the United States Government paid for the navigation locks and half the cost of the dams.)

3) Page 739-2 on privately-owned thermal power overlooks some 5,000,000 KW of TVA steam plants and more being built.
4) Page 739-3 the low feasibility ratios should explain the unfair ground rules used.
5) Page 739-11 the installation of additional peaking capacity is a good recommendation but should not be coupled with the fallacious idea that storage should be reduced. In the Columbia River Basin we need perhaps

1. Exec. Secy., Northwest Public Power Assn., Inc., Vancouver, Wash.

45,000,000 acre-ft of usable storage plus much pondage to permit flexible operation and increasing use of hydro for peaking.

6) Page 739-13. The discussion on McNary allocations and our protest thereon overlooks at least the following factors—(a) a recent Corps of Engineers analysis indicates a doubling of navigation benefits since the 1948 review of the 308 report, (b) the use of a more realistic service life for dams as permitted and recommended by Bulletin "F" of the Bureau of Internal Revenue entails the evaluation of navigation benefits not on a short term 50-year basis but on a realistic long term 150-year basis, and (c) the present McNary cost allocation fails to utilize the permissive legislation relative to allocating fish ladders (costing almost $30 million) as nonreimbursable.

Discussion

Every generation has the option and the right to overcharge itself, as for example by setting power rates so as to pay out a dam over a short term 50-year basis.

The moral position is quite different if we in this generation set such severe ground rules of criteria for hydro development that certain sites are developed on a partial basis and thereby preclude either present or future full development.

It is to the interest of both the present and of future generations to aim now at full and comprehensive ultimate development of the water resources of each river basin even though actual construction may come later.

In defining the ground rules for comprehensive development the writer would establish as reasonable those criteria which come closest to facilitating comprehensive development consistent with sound business principles. Hence I recommend:

a) Interest rate on Federal power investment should be the actual cost of money to the Federal government (average interest rate on all outstanding, marketable interest-bearing obligations of the United States Treasury).

b) Payment of no taxes on Federal power investment except (1) in-lieu-of taxes to prevent hardship caused by such plant, and (2) subsidies required by law for irrigation.

c) Use of realistic service life for dams based on FPC uniform system of accounts, and specifically the use of 150-year service life under FPC account #322. This criteria should be used for accounting purposes, depreciation and economic justification. This does not preclude the option of a 50-year basis for payout and rate making.

Finally, and in conclusion, the writer feels this topic should be reopened and given broader treatment than can be expected from an officer who is subject to the so-called partnership policy which is in such ill-repute among the people and indeed in the Congress.

These are the writer's personal observations and not necessarily those of the Northwest Public Power Association.

Discussion of "THEORY FOR THE DESIGN OF UNDERGROUND PRESSURE CONDUITS"

by D. J. Bleifuss
(Proc. Paper 741)

D. J. BLEIFUSS,[1] M. ASCE.—The writer is pleased to see the comments of Messrs. Vaughan, Nicolaou, Martin, Veltrop and Pei.

At the time the Kemano penstocks came up for design, the writer checked with other engineers responsible for the design of similar installations made or being made, and found that in each case a purely arbitrary assumption had been made as to the distribution of stresses between steel and rock. He then made an effort to devise some more or less logical basis for this distribution, to bolster his judgment in making a decision on the question.

He fully realizes the uncertainties attendant on the problem; they make quite an impressive list:

a) The original horizontal and vertical stresses in the rock are not equal; their ratio is not known, and probably differs radically from point to point.

b) The original stresses are disturbed by tunneling operations, in varying amounts and to varying distances from the tunnel.

c) The concrete shell placed between a steel lining and the rock shrinks upon setting and its contact with both steel and rock is disturbed. Grouting to cause compressive stress in the steel is not a very successful operation; anyone who tries it is going to have some very expensive repair work on partially collapsed steel lining.

d) When a steel conduit inside a tunnel or shaft is filled with water for the first time, it deflects, and so do the concrete shell around it and the rock around the shell. When drained thereafter, assuming that it has not been stressed beyond its elastic limit, the steel returns to its original dimensions. The rock and concrete, having suffered some plastic deformation and some adjustment along joint planes and cracks, do not return to their original dimensions, and the contacts between steel, concrete, and rock are impaired.

e) There are temperature differentials.

f) There are imperfections in construction.

It therefore appears that any scientifically exact solution is impossible and that the most which can be hoped for is the establishment of some guide to judgment. One thing is certain; in almost any rock at depth the rock takes a large proportion of the bursting pressure, and there is no sense in putting in a steel lining strong enough to take it all. When the engineer makes his design, he needs some idea as to how much steel he should use.

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1. Cons. Engr., Sverdrup & Parcel, San Francisco, Calif.

Paper 1017 PO 3

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

ARCH DAMS: DEVELOPMENT IN ITALY

Carlo Semenza,* M. ASCE
(Proc. Paper 1017)

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time it is not known exactly how many papers will be printed from the Symposium. So far, eleven papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by Robert E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); and "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017).

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 1017 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

* Central Manager and Chf. Engr., Hydr. Constr. Dept., Societa Adriatica di Elettricita, Venezia, Italy.

SYNOPSIS

The reasons for extensive development of arch dams in Italy are listed, and the historical stages in that development are traced, with numerous examples.

The methods of analysis are briefly reviewed and special features of construction procedure are brought out. A final section is devoted to an explanation of the benefits of the peripheral point which has been successfully used by the author in several large arch dams.

INTRODUCTION

The rather remarkable development of arch dam construction in Italy is due to several factors which are partly interconnected:

a) The geological characteristics of the country. In the Alps, which constitute the principal hydraulic power area, and still more in part of the Appennines, there is a prevalence of relatively recent formations, where fairly narrow gorges are quite common, for which the arch-dam is the most natural solution.

b) The cost of skilled and specialised labor in the construction industry is still relatively low and hence its factor in total cost is small. Materials, on the contrary, particularly cement and steel, are an important factor. The volume of concrete therefore constitutes the major element of cost.

c) It is recognised in Italy as in any other country, that given the same degree of safety, the choice of the type of dams should depend only on the economic factor. Italian engineers for this reason have been driven to prefer increasingly refined structures in order to reduce the volume to a minimum.

d) The peculiar characteristics of the Italian mentality, which is fairly individualistic, and which therefore tends to examine every problem on its own merits, and free from any preconceived set of ideas.

As a result of this attitude of mind, the principle, valid for any country, that each dam constitutes a problem in itself to be solved according to criteria free from any pre-conceived idea, has found in Italy an ideal atmosphere for its full development, sometimes indeed beyond common limits. Hence the widespread and elastic application of the most varied structural forms.

e) This general tendency, which I will call mental, has been reinforced to some extent, both by tradition and artistic environment, since the arch, from the time of ancient Rome, has been a common architectural element: thus it was both logical and natural that modern designers of hydraulic structures should use it.

f) The realization of the exceptional resistance of the arch has grown through centuries of experience in the Italian building workers whose craft has ancient traditions and deep intuition. Even for modest structures in house-building, small and slender brick arches have been used for centuries, as, for example, in Romagna and Tuscany. (Fig. 1.)

From a technical point of view, it is now considered, in short, that the overall safety of an arch structure is far higher than that of a gravity

Fig. 1

structure, as has indeed been proved in model tests to failure. In these it was seen that the collapse of dams takes place at loads which vary between 5 and 11 times the normal, the lower values mostly corresponding to failure of abutments and not of the dam itself.

In two of his contributions to the 1955 Icold Congress in Paris, the author cited some examples which illustrate the peculiar possibilities of arch structures.

In the dome structure which enjoys considerable favor in Italy, an attempt is made to realize a "sail" structure following the funicular polygon of the hydraulic loads and working in every direction with simple compressive stresses and with as uniform a value as possible: ideal limits which cannot be reached in practice, but which correspond to maximum rationality and economy.

In the arch-gravity dams—which are designed in accordance with a more complex conception because here the effect of weight is considered—there is a tendency towards structures in which the maximum compressive stresses are as far as possible of the same order whether with empty or full reservoir, and spread as evenly as possible over the whole structure. (Fig. 2.)

The Development of Curved Dams in Italy

It is not easy to summarize the history of arch dams in Italy in a few lines.

The oldest are probably the many barrages built by Government engineers for flood control purposes, or to stop debris, in the narrow mountainous valleys or for irrigation or to drive flour-mills. Perhaps the oldest of all is that of Ponte Alto in the neighbourhood of Trento. The construction of this barrage was begun in 1537 and continued with arched heightenings in the years 1611, 1748 and 1883. The height of the barrage thus reached 46.80 m. Even though the gorge is exceptionally narrow, such a height, considering the period is worthy of note. (Fig. 3.)

Naturally, these modest dams did not claim to utilize the masonry to the full; they were proportioned roughly on the cylinder formula and almost always built of stone masonry, often of cyclopean type.

Of course, the great development of arch dams has its true beginning in more recent time. A table of the arch dams existing in Italy is attached.

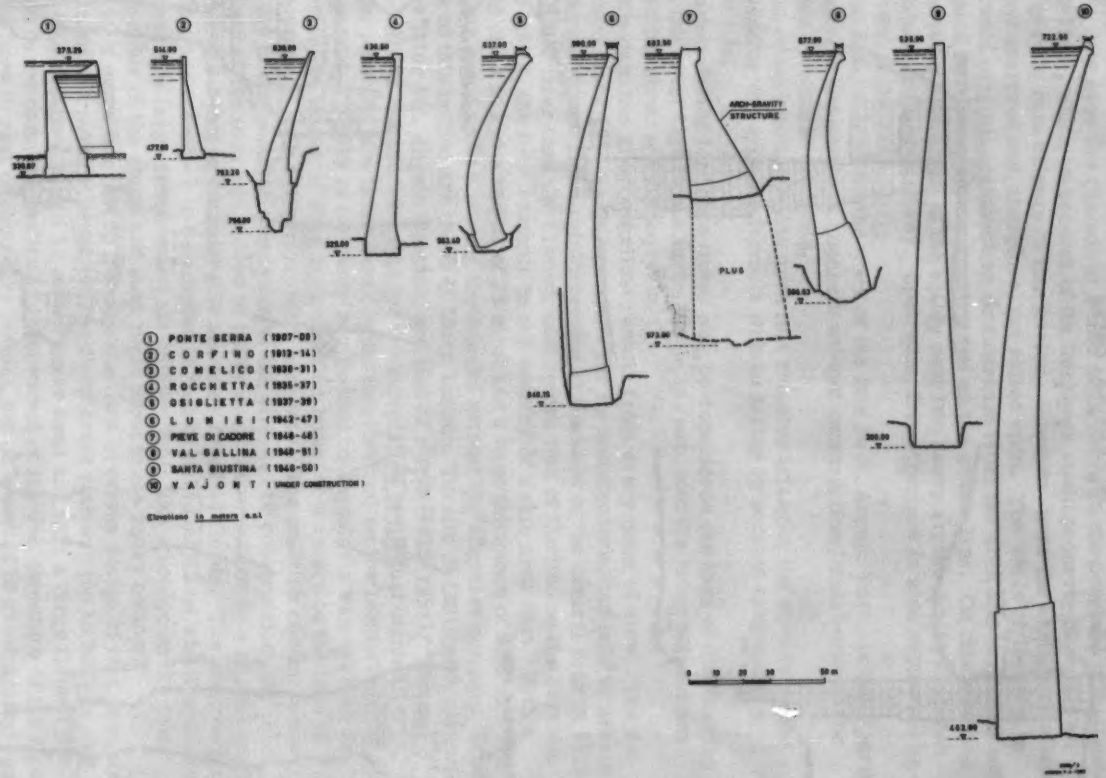
Generally speaking, it can be said that the struggle to economize has, from the very beginning, been one of the foremost idea in our constructional work and has pervaded its whole subsequent development.

In this development one can distinguish certain fundamental stages which of course cannot be exactly defined in the progress of time: that is, they are superimposed one above the other. In general, and to avoid any misinterpretation, I would point out that in my rapid summary, I shall limit myself to mentioning those constructions which have marked important steps in conception and realization: exclusion, therefore, casts no aspersion on the importance of any given structure, but is simply due to the fact that in my view that particular dam does not constitute an essential step forward with respect both to preceding and following structures.

The First Stage

The first stage, from the beginnings up to about 1930, is characterised by

SOME TYPICAL ARCH DAMS IN ITALY



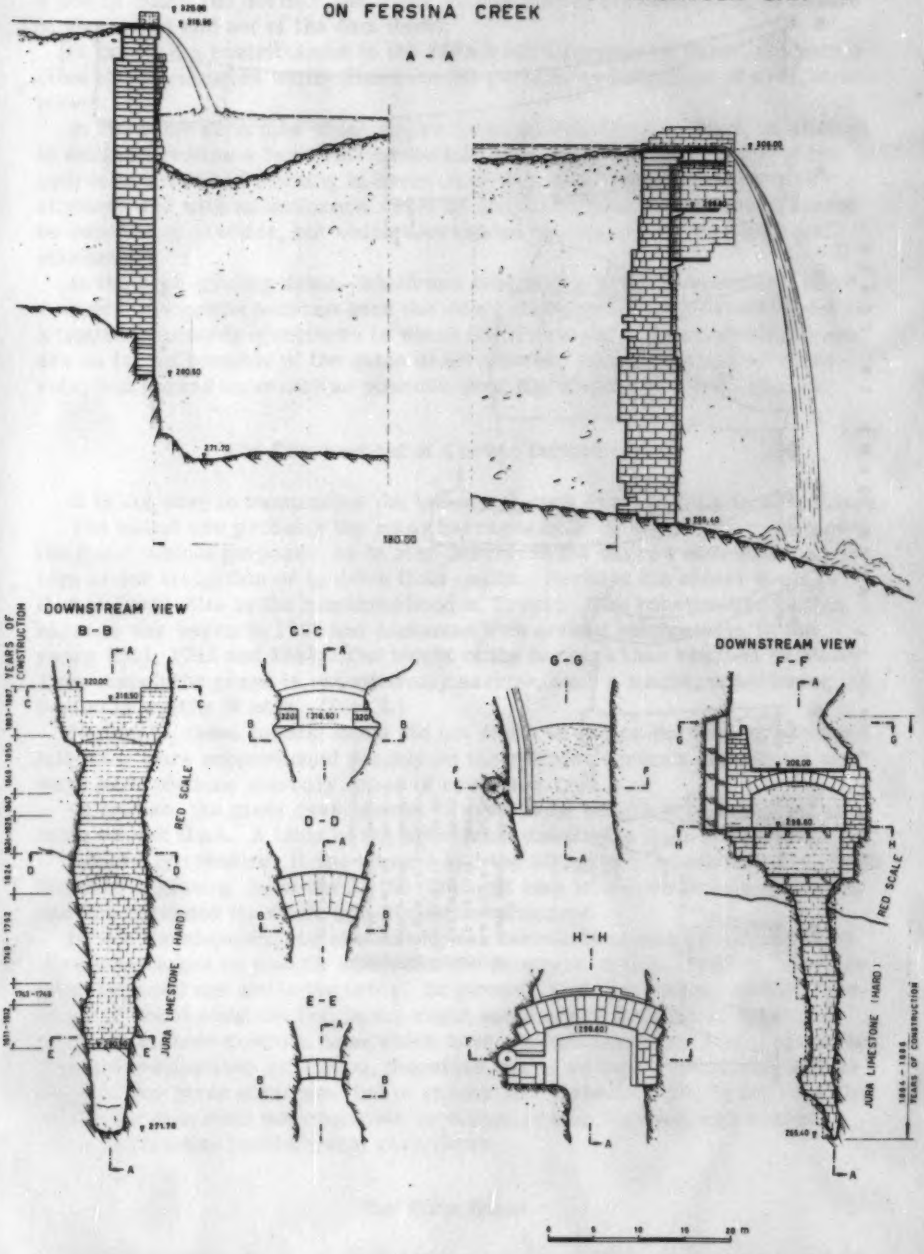
① PONTE SERRA (1907-08)
② CORFINO (1913-14)
③ COMELICO (1930-31)
④ ROCCHETTA (1935-37)
⑤ OSIGLIETTA (1937-38)
⑥ LUMIEI (1942-47)
⑦ PIEVE DI CADORE (1946-48)
⑧ VAL GALLINA (1948-51)
⑨ SANTA GIUSTINA (1948-50)
⑩ VAJONT (UNDER CONSTRUCTION)

Elevations in meters a.s.l.

ASCE SEMENZA 1017-5

a) PONTE ALTO DAM
ON FERSINA CREEK

b) MADRUZZA DAM



the construction of arch dams with generally cylindrical upstream face and vertical axis.

The dam over the Cismon at Ponte della Serra in the province of Belluno (1906-1909), which on account of its thickness could properly be called an arch gravity dam belongs to this period. However, here the cantilever effect was not analysed and, therefore, not relied upon. The whole of the dam is somewhat original inasmuch as downstream from the arch and in contact with it a thick vertical pier was constructed on the center line. On this pier and on the two rocky sides of the valley rest two heavy arches whose extrados constitutes a flood-spillway. Upstream, this structure is also supported by the crest of the arch.

The dam, which was the work of the late Prof. Angelo Forti, is built partly of masonry and partly of concrete without construction joints between the various structural elements.

The cross-section of the arch is a massive triangle; the structure was analysed by the cylinder formula without taking in account temperature variations.

Rather daring for those times may be considered the idea of discharging from the crest high floods up to 750 cu.m./sec. into the river without any protecting structure downstream.

This dam has given excellent results from every point of view. The Author who since 1929 was responsible for its maintenance, had only to arrange for partial downstream reinforcement at the base of the central pier in 1929.

A little later than the Cismon dam comes that of Corfino on the Serchio (1913-1914), the first example in our country of a thin arch dam. With a height of 35 m and a radius of 28 m, it has a base thickness of 5.50 m. The dam has stood up to a violent earthquake without the slightest damage. Other dams of this period are those of Muro Lucano, Turrite di Gallicano, planned by the later Mr. Angelo Omodeo, and that of Furlo (1919-1921). The so-called Ritter method was used for the calculation of the last-named.

The planning of a large dam (the Sottosella dam) on the Isonzo built by the Author in 1937-1939 in territory now passed to Jugoslavia was carried on conservative lines: it does not represent however a return to the idea of massive dams with cylindrical upstream face. The solution chosen is due entirely to the peculiar requisites of the site and of construction; the main problem of the dam was indeed to carry out the foundations in a narrow rocky river-bed subject to considerable and very frequent floods. The structure of the dam had therefore to be adapted to that of the foundations which consisted of one single large compressed air caisson built like a bridge on the two rocky sides of the gorge and sunk right down to the rocky bottom.

It is of some interest to mention that in a previous design of this dam, as I explained in a publication of 1940, a flood crest spillway (up to 2000 cu.m./sec.) was designed with a 12 meter water head and with a large flume connected to the dam, but statically independent of it. The structure of the flume did not have to rest on the rocky bottom, but on a series of great arches spanning the rocky banks.

The Second Stage

The last ten years of what I called the first stage, that is to say, the years after 1920, witnessed the adoption of the method of analysis based on the

theory of horizontal arch elements with encastrement at the abutments which were held to be "rigid." Prof. Guidi was the first exponent of this in Italy. At the same time the principle of the variability of radii was applied tending towards the type with a constant or nearly constant central angle of the arches.

The characteristics of the arches had to be developed rapidly: the vertical cross sections pass from triangular forms more or less thick at the base to curved profiles, sometimes with the upper arches considerably overhanging downstream at the crest. In other words, the simple arch begins to tend towards the double curvature structure. The plan consequently becomes more complex and tends towards a shell shape which later on will be characteristic of dome dams.

There is also a trend towards the principle of the variability of thickness in the individual arches with appropriate thickening from the crown to the abutments.

The first important example of this type of structure is the Comelico dam on the Piave downstream from S. Stefano di Cadore (1930-1931), which, like the dam at Fortezza of which I shall speak later, was the work of the late Mr. Nicolai, who left a notable mark on the history of arch dams in Italy.

The dam is 66 m high at the deepest point of the foundation, 1.20 m thick at the crest, 8.66 m at the base. By applying the principle of the constancy of the central angle of the arches, a cross section was obtained with an upstream face which overhangs 17 m downstream, and a downstream face which overhangs 10 m. The result was a construction of some elegance. The dam is built of concrete with light steel reinforcement for a better distribution of the stresses and a heavier reinforcement at the abutments, to put into practice the theoretical encastrement conditions.

Practically contemporary (1931) is the Ceppo Morelli dam on the Anza, which was the work of a great Italian engineer, Mr. Vincenzo Ferniani.

In the Rocchetta dam, of slightly later date (1935-1937) even the central angles of the arches vary (from 100° to 120°); the extrados radii decrease from 71.90 m at the crest to 33.90 m at the base. The upstream face has an overhang of about 17% and the downstream one of 5%. The arches thicken slightly from the crown to the abutments. Particular attention was given to the surface of support of the structure in order to render it practically continuous according to the principle of the perimetral joint adopted later (see Osiglietta dam). This dam has also undergone the test of some earthquakes without suffering any damage whatsoever. (Fig. 4.)

The Fortezza dam on the Isarco (1938-40), i.e. slightly later than the Osiglietta dam of which I shall speak further on and which marks the beginning of a definite tendency towards the dome structure, has a steep downstream overhang; radii, central angles and thickness of the arches vary continually. The arch (with overhang lip) has no encastrement and rests both at the abutments and at the base on a sort of continuous cradle, as in the case of Osiglietta.

The Third Stage

All these dams may perhaps be considered as transitional structures leading towards the more refined type, now widely used in Italy, of the dome dam. The first, and remarkable example, is that of Osiglietta in the Appennines of

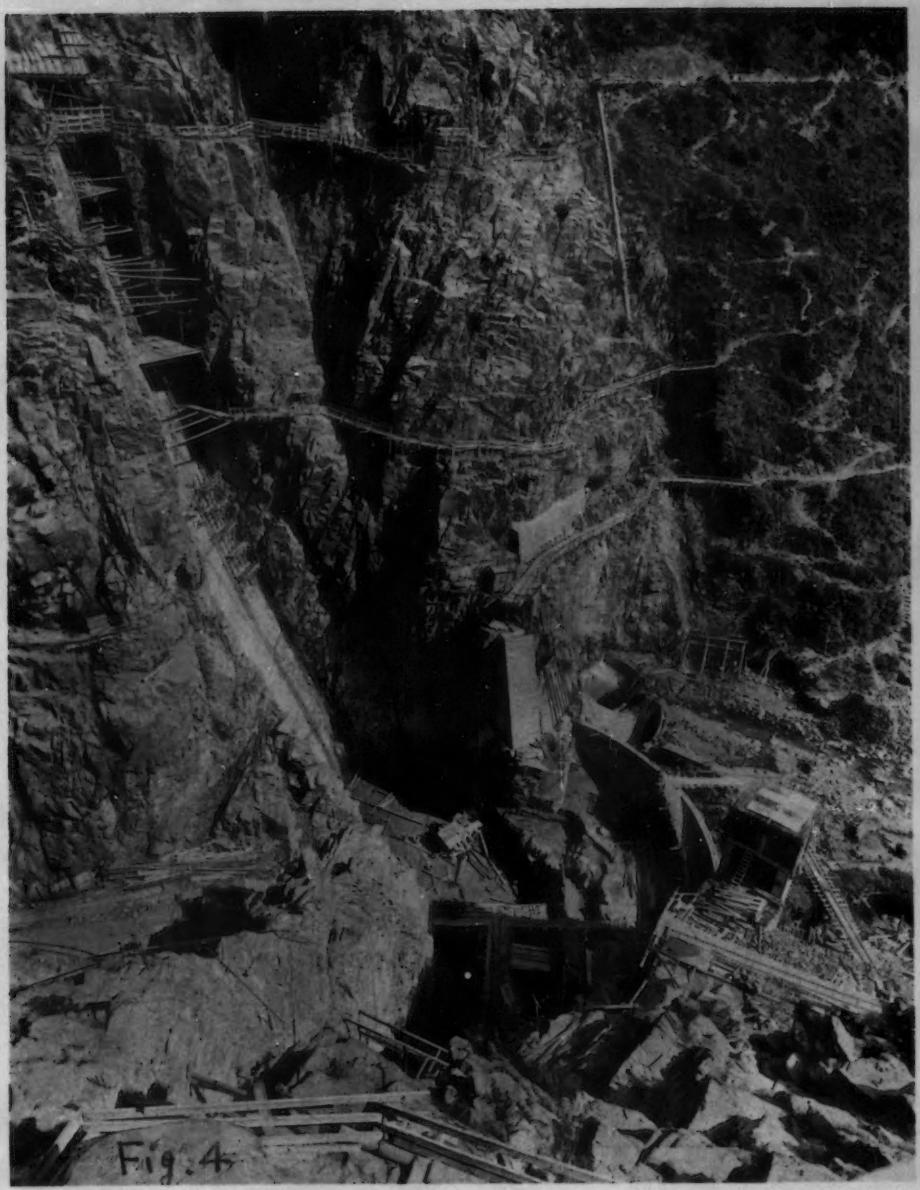


Fig. 4

Liguria (1937-39). About 67 m high with a crest length of 224 m and a volume of 75,000 cu.m., it has many characteristics which are still being discussed in this field. It consists of horizontal elements of varying thickness with downstream face radii of 21 m at the base to 101 m at the crest and with arch central angles of from 79° to 129°. The crown cross-section has an upstream overhang at the base of 2.40 m and a downstream overhang at the crest of 15.45 m; both faces are strongly curved. The dam is perfectly symmetrical and its abutments have practically a continuous curvature. (Fig. 5.)

Another noteworthy characteristic is provided by the suppression of the lateral encastrement: the dam simply rests on a sort of continuous concrete saddle which follows the rocky surface of support. It constitutes the first example of this device, of which I shall speak fully in the last part of this Paper.

The Osiglietta dam was followed almost immediately, i.e. already during the second world war, by the excavations and the first concrete pouring for the Lumiei dam which was planned in 1939-40 and completed in 1946-47; and after the war by a considerable number of dome dams of great height. These dams were built in sites, and therefore with characteristics, very different one from another, but nevertheless designed and carried out along fairly similar lines. In the region of the Tre Venezie alone there are a score of structures of this type.

The Lumiei dam (1942-47) is doubtless the most important of this group: 136.15 m high with a crest length of 138 m, volume 100,320 cu.m., it is one of the thinnest structures of its kind. (Figs. 6 & 7.)

It was for some years the highest arch dam in the world and was the first dam in Italy whose analysis began to take account of the effect of rock deformation.

Another dam built in that period is especially remarkable for some details of its abutment-saddle of which I shall speak in the last part of this paper: that of Val Gallina (1949-1951). (Figs. 8 & 9.)

Outstanding for its peculiar site conditions, characteristics and height, is the dam of S. Giustina on the Noce (h=152.50 m, V=120.000 cu.m.) which in its day, like that of Lumiei, held the "blue ribbon" for arch dams in the world, an honour now held by Tignes.

The characteristics of the gorge at S. Giustina, with practically vertical sides, obviously required a solution in which the small variation in the central angles and the almost vertical upstream face made it resemble the cylindrical type very closely. (Fig. 10.)

The dam of Rio Freddo in the Western Alps near Cuneo is peculiar because it consists of a thin arch supported by two large concrete shoulders (height 45 m) which sustain the pressure of the arch. From the two artificial abutments two short gravity walls keep the water upstream.

The construction of the large Vajont dam in a very narrow and striking limestone gorge in the Dolomites has started this year. With its height of 262,50 m, a chord of 160 m, V = about 335.000 cu.m., it will rank amongst the highest dams in the world.

The Fourth Stage

In the last fifteen years the problem of arch-gravity dams has been squarely faced in Italy. Many dams built in the past have actually utilized,

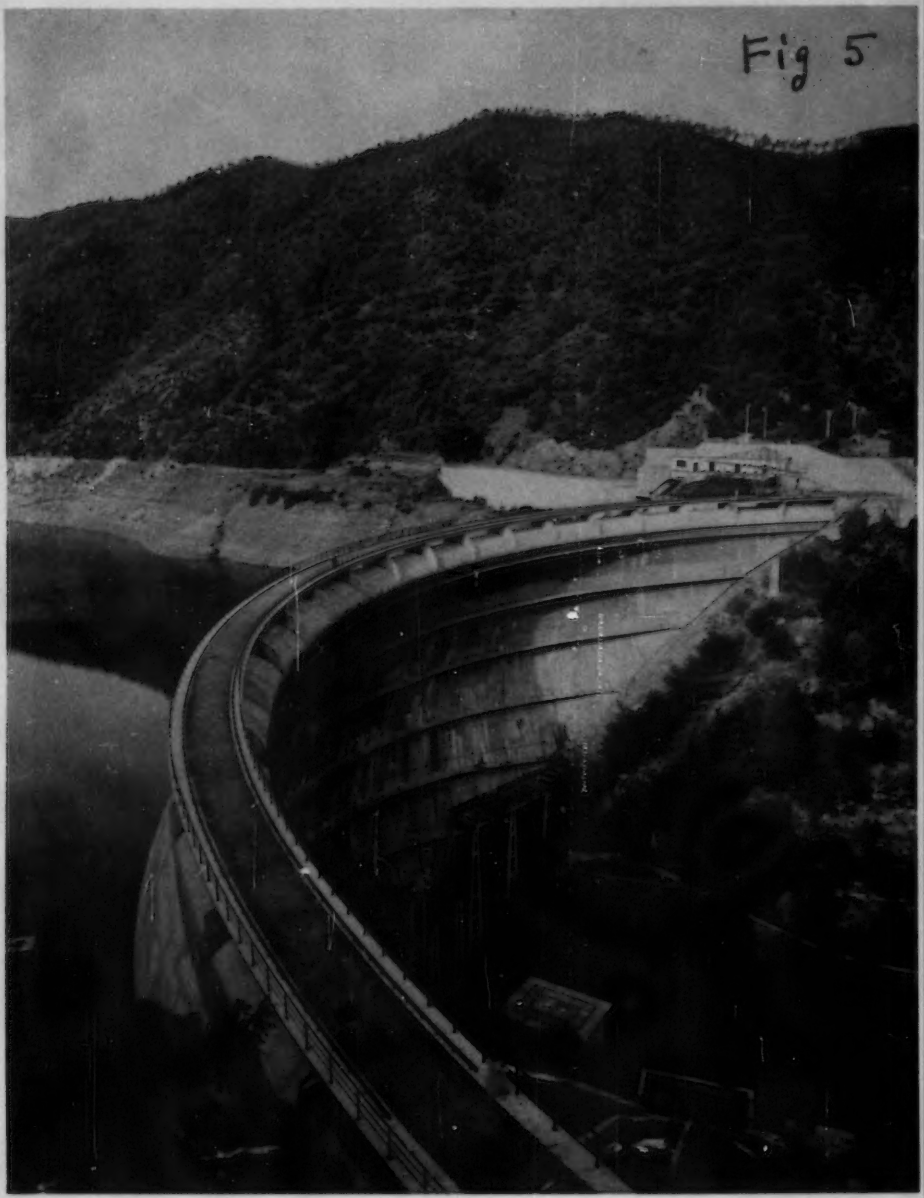


Fig 5

Fig 6
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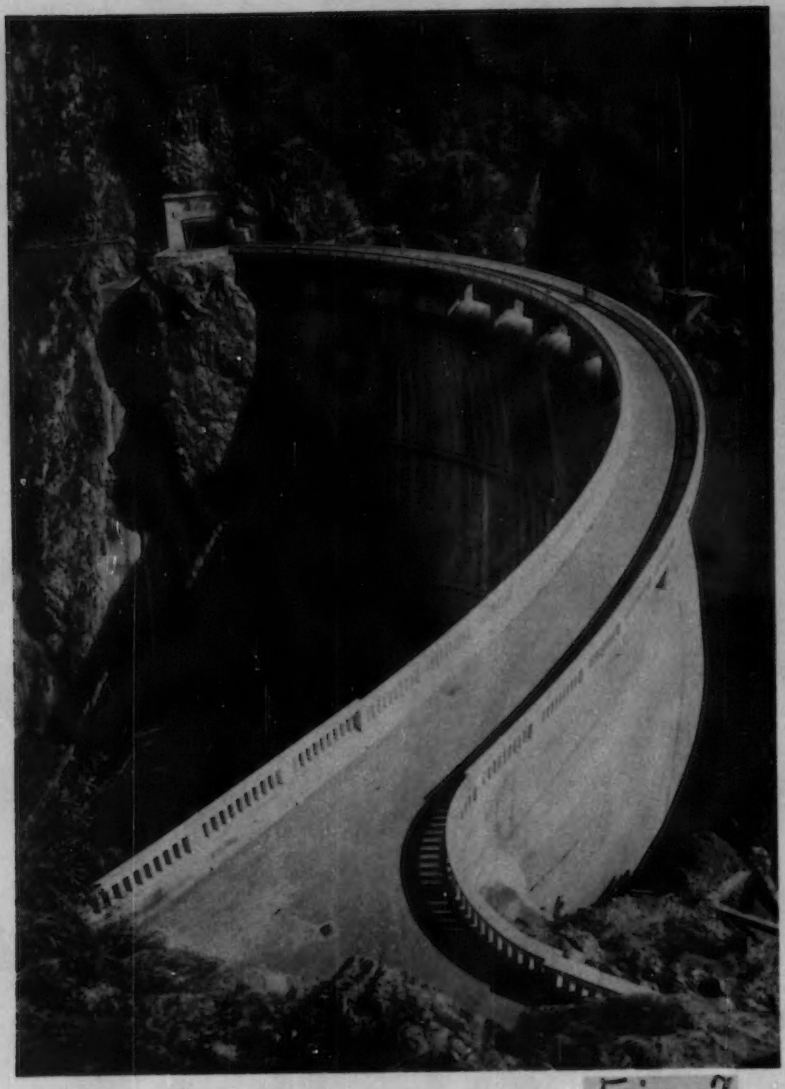


Fig. 7

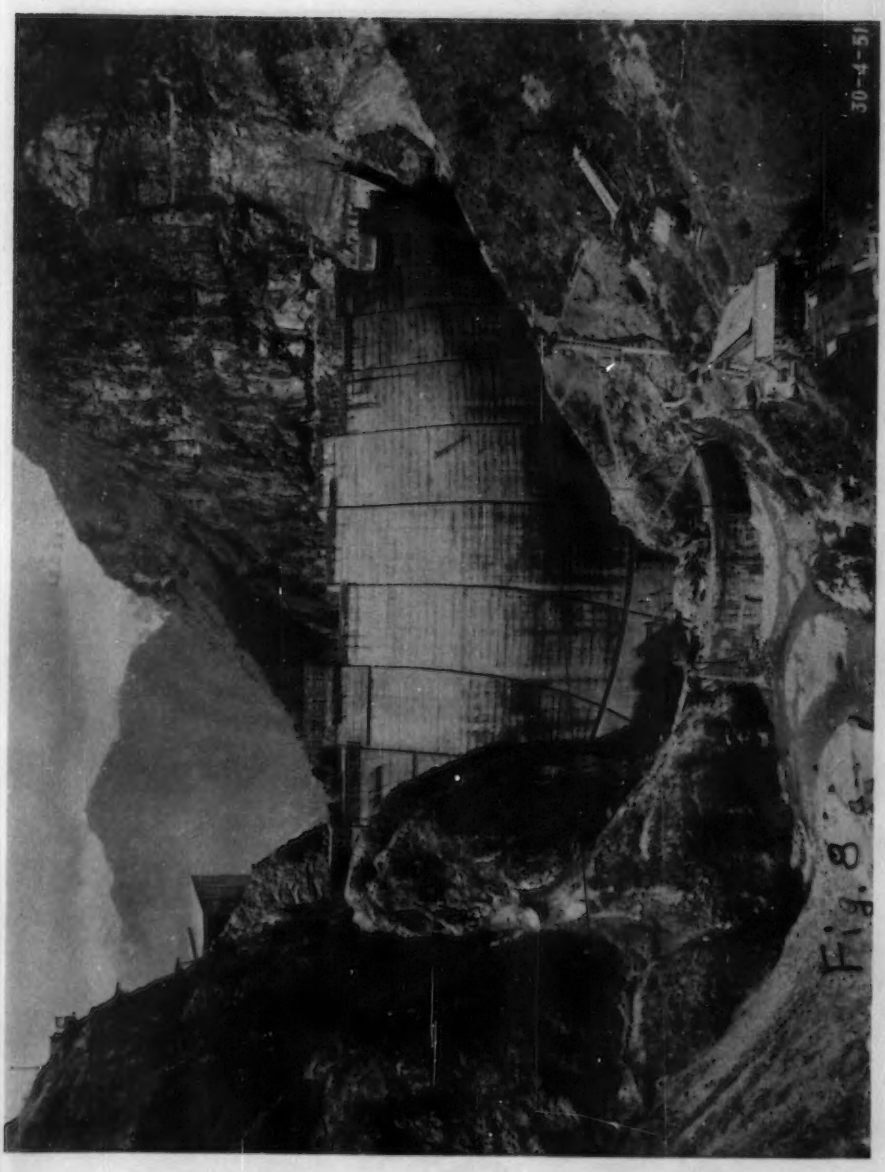
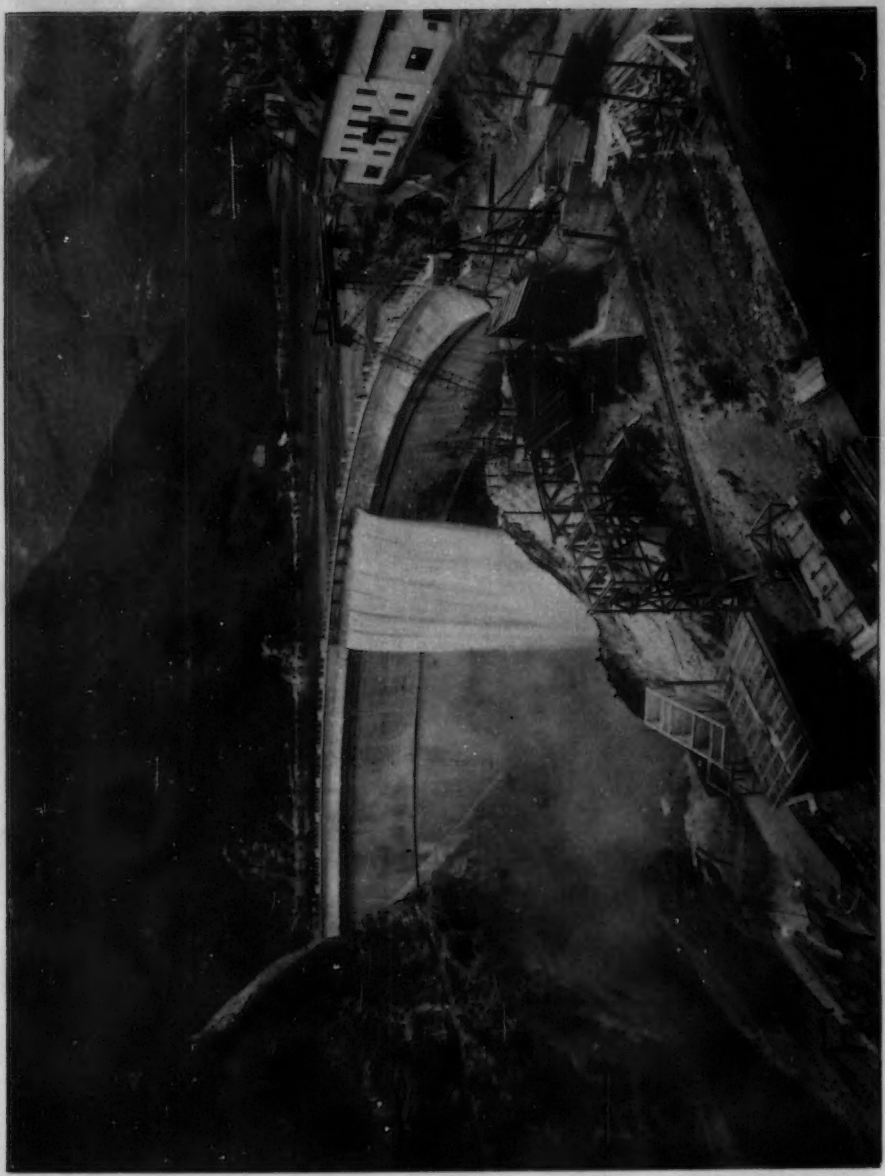


Fig. 8



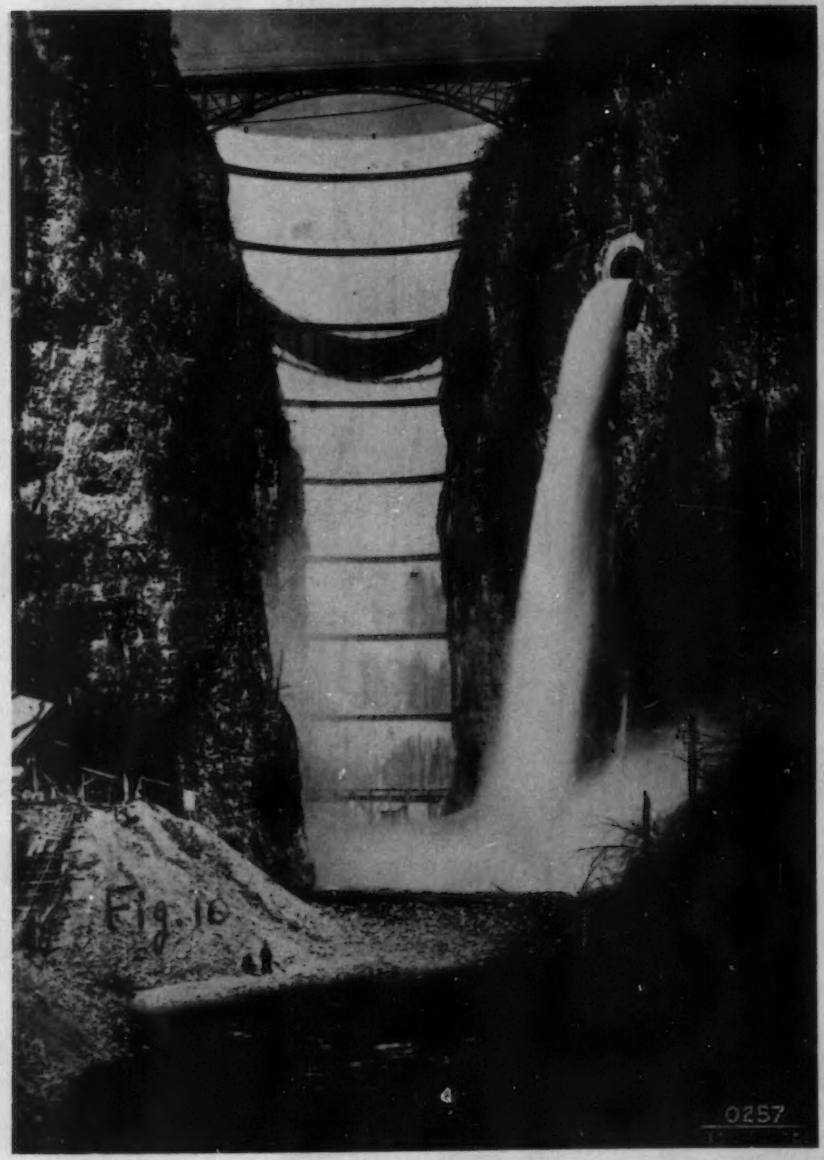


Fig. 10
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because of their characteristics, the double way of working of the structure, but the conception in which also the cantilevers—that is the weight—come to be considered as a determining static element, was adopted, I believe I am right in saying, for the first time in Italy in the Pieve di Cadore dam. (Figs. 11 & 12.)

As regards the behavior of the dam as a dome, in this particular case, owing to the features of the site, it is of secondary importance; on the contrary the other great arch-gravity dams built or in construction since 1945 tend to exploit the dome function to the utmost.

The Pieve di Cadore dam is really a unique example, both for its dimensional ratios and for the fact that its arched structure (of constant height on the perimetral joint) rests for the most part on a rocky plateau, while the remainder is supported by a concrete plug which closes a deep narrow gorge lying alongside the plateau.

The dam has a maximum height (from the base of the plug) of 111.50 m; the average height of the arch gravity structure on the general foundation line (elev. 634 m) is 55 m, the crest length 410 m. The total volume is 377,000 cu.m. The thickness varies from 6 m at the crest to 26 m at the base of the arch and 36 m at the base of the lateral foundation plug. The arches up to elevation 640 m are fundamentally circular with a constant radius for the upstream face, whereas the downstream face is polycentric; above 640 m the axis gradually becomes polycentric with more and more substantial differences between the radii. Hence the cross sections vary continuously from the crown to the abutments; they bend downstream at the crown and straighten out at the abutments as in dome dams.

The Pieve di Cadore dam is also remarkable for the number of measuring devices installed.

Following the construction of the Pieve di Cadore dam, other noteworthy arch-gravity structures were built. Among these we will cite, especially, the dams of Publino in Valtellina (h = 34 m, chord length 173 m, volume = 34.000 cu.m.), Travignolo (h = 110 m, volume = 260.000 cu.m.), Mucone (h = 55 m, volume = 62.500 m), Fiastrone (h = 87 m, volume 106.000 cu.m.; moreover, those at present under construction at Beauregard in Val d'Aosta (h = 132 m, volume = 440.000 cu.m.), Cancano in Valtellina (h=173 m, volume = 1.100.000 cu.m.), Mulgargia (h = 99 m, volume = 240.000 cu.m.) Flumendosa (h = 119 m, volume = 305.000 cu.m.) in Sardinia, Frera in Valtellina (h=138 m, volume = 420.000 cu.m.). (Figs. 13 to 18.)

Several of the dams built in the last ten years are particularly interesting because of the peculiar problems solved in connection with construction on not altogether reliable rock. Among those I have already quoted is the Val Gallina dam, the foundation of which had to a great extent to be substantially reinforced. I still remember, in connection with the Beauregard dam already mentioned, that at the foot of the left abutment of this large dam there was a sort of enormous pocket of alluvial sand about 30 m high (which was discovered only after the main part of the excavation was carried out), the filling up of which required considerable concreting.

I have cited these problems not so much because they involve particular solutions from the point of view of construction but to show how in Italy in many cases there is no hesitation in adopting the arch structure even in not altogether favorable geological conditions, always of course provided the appropriate measures are taken. The study of the measures and of the special precautions to be taken in these cases has reached a pitch of development in my country which I consider worthy of interest.

Fig. 11

PIEVE DI CADORE DAM
ON THE RIVER PIAVE

DEVELOPED ELEVATION
ALONG THE MIDDLE FIBRE OF THE ARCHES

FROM DOWNSTREAM

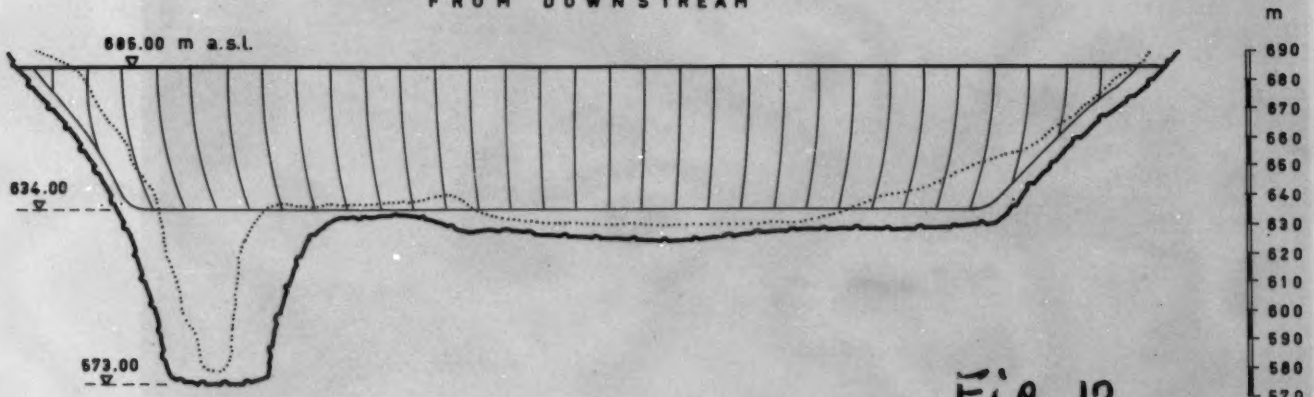


Fig. 12

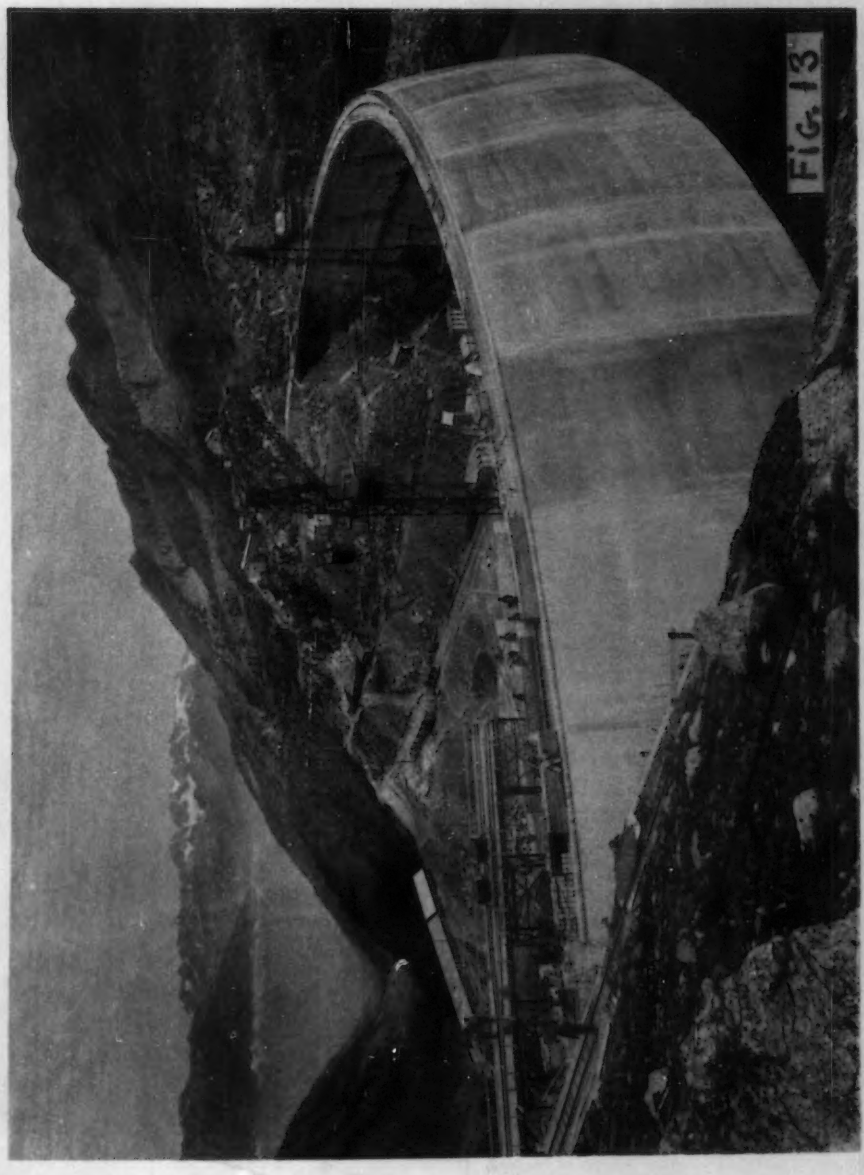


FIG. 13

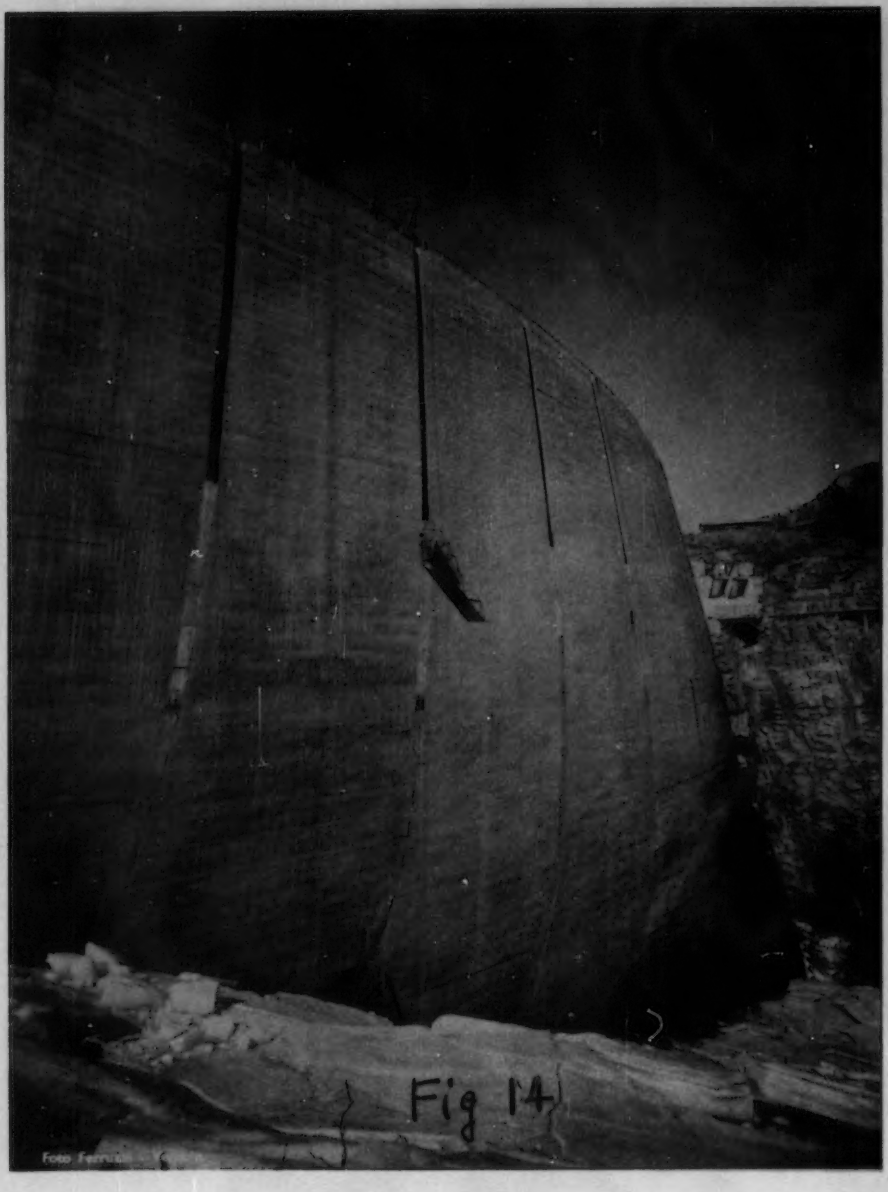


Fig 14

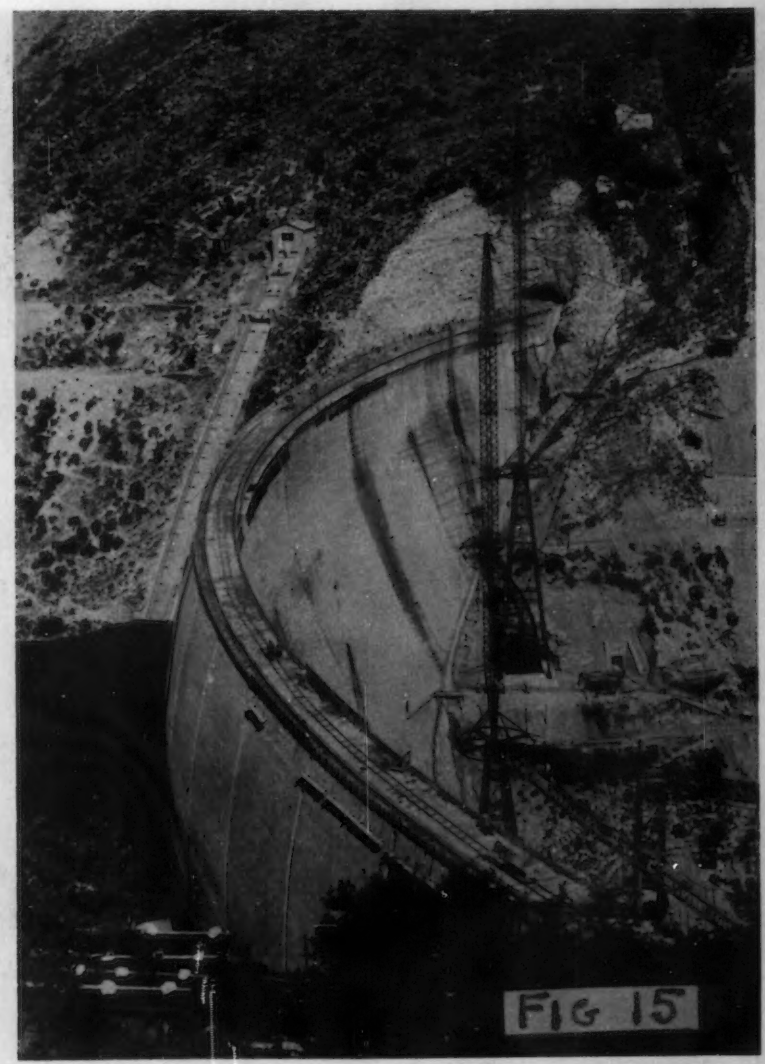


FIG 15

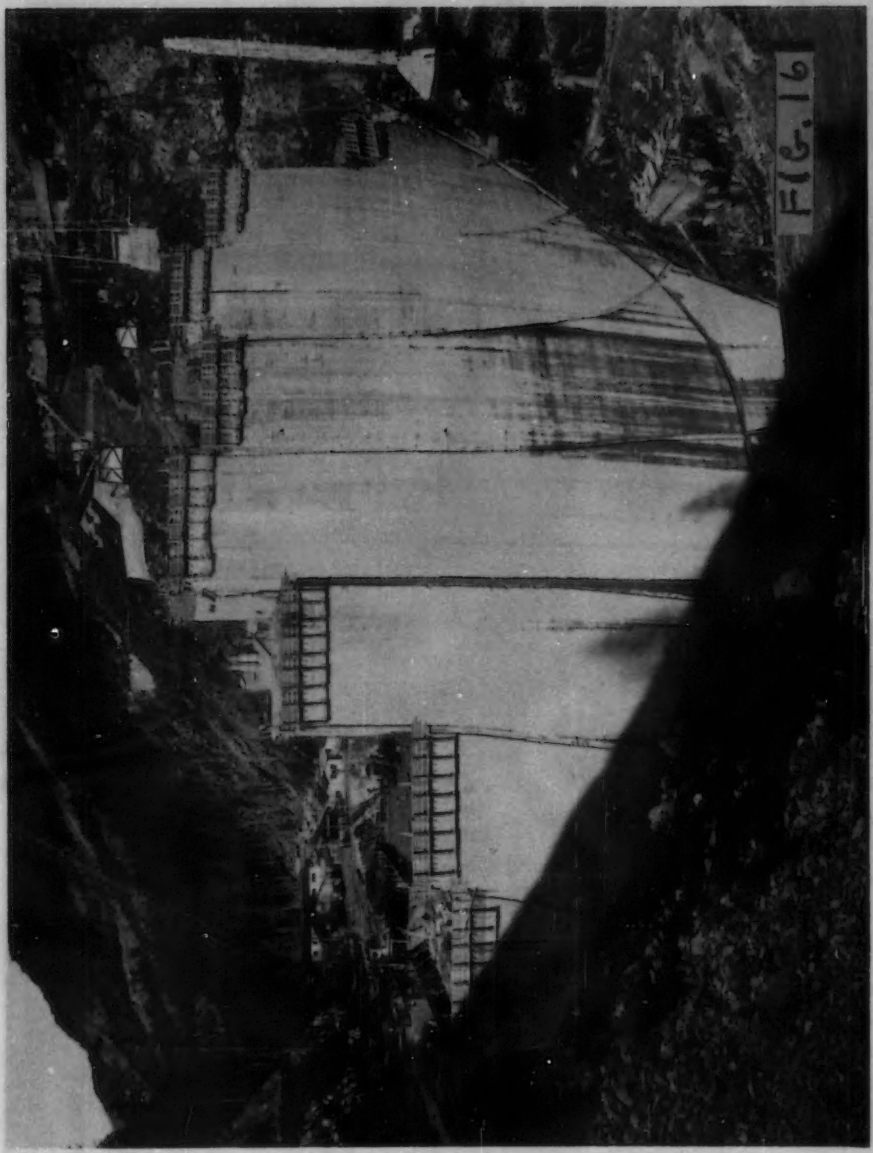


FIG. 16

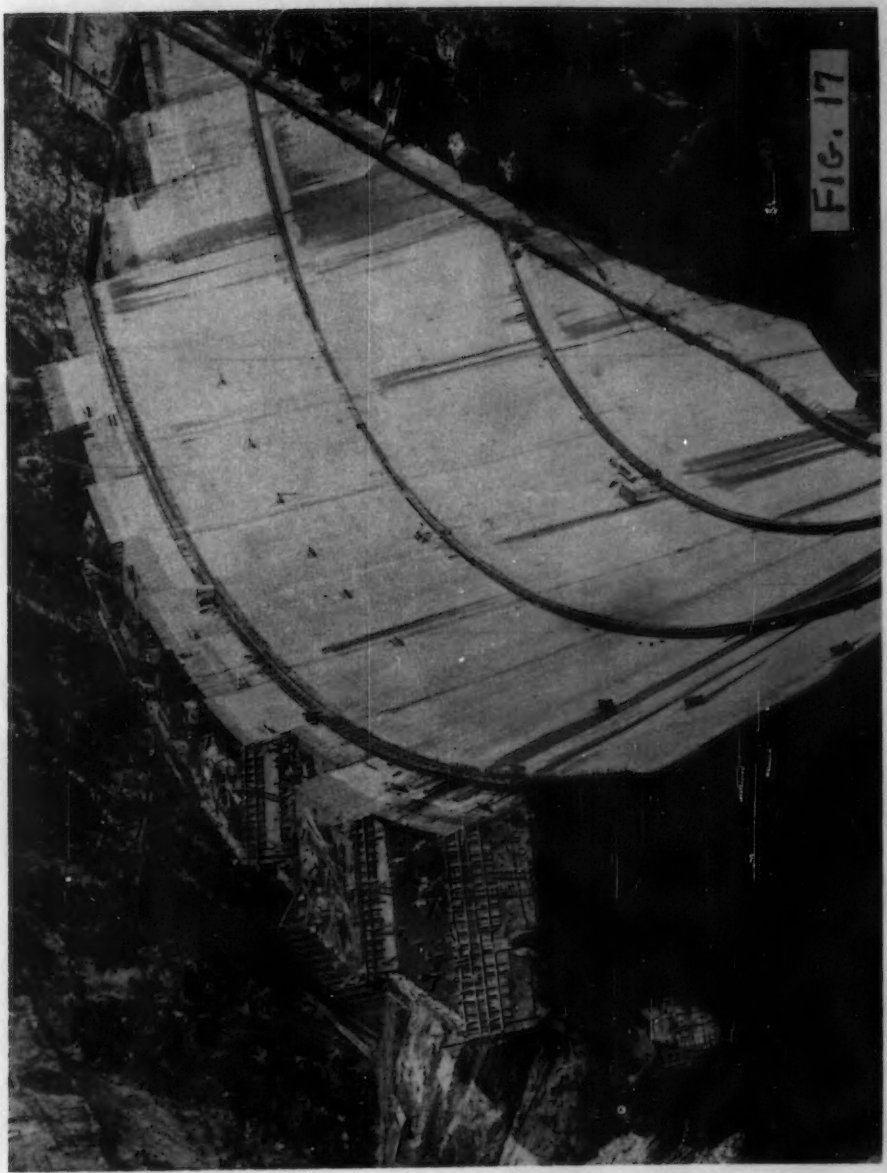


FIG. 17

FIG 18

Methods of Analysis

As I have mentioned incidentally in preceding paragraphs, the principles underlying the analysis for arch dams in Italy have followed more or less the same evolution as in other countries. The basic calculation, which even today gives the very first indications for a preliminary sizing-up is that based on the cylinder formula; assuming correlatively rather low stresses.

The Italian "Regolamento Dighe" itself (Dams Regulation) also allows the application of the cylinder formula for the lower parts of the structures, where the thickness of the arches is too great in relation to the radius to permit the use of elastic arch analysis. For all the other parts of the structure the use of this last type of analysis is fundamental. As I have already said, it was introduced by Prof. Guidi with subsequent applications by means of the ellipse of elasticity. The Italian regulations sanction this practice: the "Regolamento Dighe" in force accept in the main the elastic arch analysis for arch and dome dams which can be reasonably considered only from that point of view.

In structures where the ratios between the dimensions cause the cantilevers to have an actual static function (but not so remarkable as to make it necessary to consider them genuine arch-gravity structures), an ordinary analytical check of the cantilevers for the effect of their own weight is required. In some cases the study is limited to the central cantilever, following the Ritter and Stucky systems, in other cases extending the calculation to other cantilevers.

Furthermore, in some cases, the behaviour of the upper parts of the structure is checked considering the inclined arches (the "arches plongeants" of the French engineers): however, this control does not always give satisfactory results because the choice of the arches is often difficult to define. In other cases, the analysis with the "membrane type" calculation is adopted, i.e. an extension to thin revolution arches of the classical method by Schwedler, reported also by Krall.

The foregoing refers of course to the true arch or dome dams, relatively slender, in which the resistance of the cantilevers can be totally or partially ignored. When this conception does not apply, recourse is made to the calculation of arch-gravity structures according to the classic methods of Tölke and the Trial Load.

Special applications of these methods have been carried out by Italian specialists. Professors Danusso and Oberti have availed themselves of the Tölke method, adding some original ideas of their own. Prof. Arredi, in his turn, for the planning of the Piave dam, studied special methods (inspired partly by some ideas of Smith) which were later generalised and amplified and applied to the analysis of many other structures. Prof. Tonini used the Trial Load Method repeatedly and introduced simplifications and additions of considerable practical interest, substituting research by systems of equations for that by trials.

As a consequence of the improved knowledge of the distribution of stresses in arches, due to theoretical studies and model tests, the monocircular arch form and the constant thickness have now been completely abandoned. Nowadays the normal form for the arch has increasing thickness and increasing curvature towards the abutments (the fillet arches), sometimes with local points of variation as in the case of polycentric curves, and sometimes gradually as in the case of special curves.

The hypothesis of foundation deformability has also been applied for the first time in Italy by Oberti in 1938 according to widely known principles and formulas to be found also in American technical literature.

A great deal of attention has been paid to the problem of the stresses due to temperature variations and there are important studies on this subject by Professors Guidi, Puppini, Ippolito and Arredi.

The widespread use of model tests, which are genuine tools for three-dimensional calculation of structures, has been a great help in the study of arch dams in Italy. Prof. Oberti will deal with this. I will therefore confine myself to mentioning the fact that model tests permit the analysis of even very special circumstances of great interest for the study and adoption of arch structures, such as peculiar conditions of the rock of the abutments, the influence of assymmetry and now also the behaviour of the structure in case of earthquakes of all kinds and degrees.

With regard to the maximum stresses which can be used for concrete there is a tendency to accept values equal to one fifth of the 90 day strength the regulation in force still prescribes 1/5 at 28 days and 1/7 at 90 days but it will probably soon be modified. In Italy the normal values reached for arch and arch-gravity dams are of the order of 60 to 70 kg/sq.cm. for the compressive stresses and 8 ÷ 10 kg/sq.cm. for tensile stresses.

To conclude this short chapter on the principles of calculation I would like to place the objective view point of the Servizio Dighe (Dams Department) of the Italian Ministry of Public Works which, without ever departing from its safety standards, has allowed every rational step in our field.

Construction Methods

I do not think that the principles of construction in Italy differ greatly from those practised in other countries. We may consider before all the question of the time required for the construction. In particular, one may say that in Italy the construction of an arch-dam is not held to constitute, from that point of view, a more serious or worrying problem than that for another type of concrete dam.

In other words, it is held that a dam, simply because it has to be built as an arch dam instead of a straight one, cannot make the whole construction a longer business.

The difficulties here might arise exclusively from the climatic conditions which in many Alpine districts are peculiarly severe. Hence the arch solution, with its lower volume, may with suitable organisation involve less time. From a monthly pouring of 44,150 cu.m. for many consecutive months on the Piave dam in the years 1948 and 1949, the figure of 54,700 cu.m. was reached in 1955 for the Cancano dam in the Upper Valtelline, and those figures were almost reached on Pian Telessio and Beauregard dams.

It is obvious that, in case of necessity, for the arch-gravity dams of great thickness, the daily pouring could be increased so as to reach quantities comparable with those of a gravity dam at least in proportion to their respective thicknesses. The arrangement of radial blocks, normally 10 to 15 m thick in Italy, corresponds exactly to the pouring for blocks in gravity dams. Obviously the difficulties of bigger concrete pourings do not lie so much in the lesser thickness as in the height of the layer that can be poured daily, and this is true in general for any type of dam. There may be further limitations

TABLE 1. SIMPLE CURVATURE ARCH DAMS

a) with mostly constant radius
b) with mostly constant angle

Name of the Dam	Type of the Dam	River Basin	Dam	
			Maximum Height m	Volume $m^3 \times 10^3$
1. Crosis	a)	Isonzo	38,20	-
2. Ponte della Serra	a)	Brenta	44,40	19,7
3. Corfino	a)	Serchio	37,50	2,20
4. Muro Lucano	a)	Sele	53,25	10
5. Turrite di Gallicano	a)	Serchio	42,00	9,05
6. Lago Sucotto	a)	Serio	15,20	0,43
7. Ogna Superiore	a)	Serio-Adda-Po	26,60	-
8. Ogna Inferiore	a)	Serio-Adda-Po	11,50	-
9. Gurzia	b)	Dora Baltea	50,00	8,00
10. Val Molinara	a)	Pioverna-Adda-Po	17,00	-
11. Valla (L.Scuro)	a)	Bormida	47,00	23
12. Lago Campelli	a)	Serio	20,00	1,24
13. Rimasco	a)	Sesia-Po	30,50	-
14. Tul	a)	Tagliamento	40,10	1,98
15. Orichella	a)	Neto	36,20	9,50
16. San Colombano	a)	Avisio-Adige	19,00	-
17. Ceppo Morelli	b)	Toce (Ticino)	46,00	10,50
18. Comelico	b)	Piave	66,50	31,50
19. Moledana	b)	Adda Alpina	42,60	12,8
20. Malciaussia	b)	Stura di Viù	30,50	7,30
21. Rocchetta	b)	Magra	76,00	49
22. Sottosella	a)		57,75	24,35
23. Giaredo	a)	Magra	27,50	1,80
24. Panigai	b)	Adda	41,50	5
25. Provvidenza	b)	Vomano	52,20	70,8
26. Novarza	a)	Tagliamento	38,75	1,35
27. Santa Giustina	b)	Adige	152,50	112
28. Ganda	b)	Adda Alpina	30,00	6,00
29. Zolezzi	a)	Sturla-Entella	19,00	

Reservoir		Year built	Bodies to which the Dam belongs	
Storage Capacity $m^3 \times 10^6$	Max.Level m.a.s.l.			
0,150	259,15	1901	Filatura Cascami Seta	
4,00	379,25	1907-1909	Sade	Overflowing
0,87	514,00	1913-1914	Selt-Valdarno	
5,78	567,00	1914-1917	Soc.Lucana I.I.	
0,89	298,00	1915-1916	Selt-Valdarno	Overflowing
0,45	1.863,50	1919-1923	Az.El.Crespi	
0,150	-	1922	Villa Carlo e F.lli	
0,150	-	1922	Villa Carlo e F.lli	
1,249	427,50	1922-1925	Ovesticino	In reinforced concrete--overflowing
0,006	-	1923	Soc.An.Orobia	
2,89	280,00	1923-1925	A.F.L. Falck	Overflowing
0,47	2.044,15	1924	Az.El.Crespi	
0,200	887,50	1925	Soc.Idr.Valsesia	
0,80	268,10	1925-1928 1947-1949	Soc.El. del Tul	Overflowing
0,20	795,90	1926-1928	Meridionale	In reinforced concrete--overflowing
0,600	-	1928	Soc.E.Jacob	
0,47	780,75	1929	Montecatini	
2,08	830,00	1930-1931	Sade	
0,11	909,00	1930-1931	A.F.L. Falck	Overflowing
1,15	1.805,00	1932-1933	Ovesticino	In reinforced concrete--overflowing
5,00	403,50	1935-1937	A.F.L. Falck	
9,60	153,00	1937-1939	Sade	In reinforced concrete--overflowing
0,12	362,00	1940-1941	A.F.L. Falck	Overflowing
0,12	704,00	1941	Orobia	Overflowing
2,00	1.060,00	1941-1943 1946-1947	Terni	Overflowing
-	997,65	1946-1947	Sade	Overflowing
182,81	530,00	1946-1950	Edison	
0,071	913,00	1947	A.F.L. Falck	Overflowing
0,061	352,00	1923	Cons.Idr.Monte Aiona	

TABLE 2. DOUBLE-CURVATURE (DOME) DAMS

Name of the Dam	Type of the Dam	River Basin	Dam	
			Maximum Height m	Volume $m^3 \times 10^3$
1. Osiglietta	D.-C. O.	Bormida	76,80	75,00
2. Fortezza	D.-C. O.	Isarco (Adige)	63,50	15,00
3. La Maina di Sauris	D.-C. O.	Tagliamento	136,15	100,32
4. Bau Mandara	D.-C. O.	Alto Flumendosa	20,30	1,30
5. Monte Zovo (Avandiga)		Piave	28,40	1,43
6. Ponte Racli	D.-C. O.	Livenza	75,35	18,00
7. Valle di Cadore	D.-C. O.	Piave	61,25	4,607
8. Val Gallina	D.-C. O.	Piave	92,37	99,10
9. Pezzè di Mbena	D.-C. O.	Avisio	28,30	1,50
10. Barrea	D.-C. O.	Sangro	62,75	4,5
11. Carboi		Carboi	47,00	32,00
12. Isolato	D.-C. O.	Mera	38,50	6,50
13. Senaiga	D.-C. O.	Senaiga-Cismon--Brenta	68,00	-
14. Corlo	D.-C. O.	Cismon-Brenta	71,00	-
15. Barcis	D.-C. O.	Cellina-Meduna--Livenza	50,15	9,00
16. Grotta Campanaro	D.-C. O.	Melfa-Liri-Garigliano	49,00	5,40
17. Rio Freddo	D.-C.	Stura di D-Rio Freddo	40,5	32,2
18. Forra dei Camini(Stramentizzo)	D.-C.	Avisio	63,00	24,0

Reservoir		Year built	Bodies to which the Dam belongs	
Storage Capacity $m^3 \times 10^6$	Max.Level m.a.s.l.			
13	637,00	1937-1939	A.F.L. Falck	
3,35	722,50	1938-1940	Ferrovie Stato	
73	980,00	1942-1947	Sade	
0,31	803,30	1942-1948	Soc.El. Sarda	
-	-	1946-1949	Sade	
25	313,00	1948-1951	Snia Viscosa	
4,90	706,50	1949-1950	Sade	
6,40	677,00	1949-1951	Sade	
0,46	1.197,00	1950	Trentina di El.	
24,30	973,00	1950-1951	Com.Sangro-Sme-Terni	
35,5	179,00	1950-1951	Ente Rif.Agr.Sic.	
1,76	1.246,80	1951-1952	Edison	
5,750	404,00	1954	S.I.I.A.	
42,600	268,00	1954	S.I.I.A.	
21,990	402,00	1954	Sade	
0,394	783,00	1954	Soc.Alto Liri	
0,325	1.202,50	1954-1955	Edison	
10,000	789,50	1954-1956	Soc.Ind.Trentina	
				LEGEND : D.-C. = Double-curvature.
				O. = Overflowing

TABLE 3. ARCH-GRAVITY DAMS

Name of the Dam	Type of the Dam	River Basin	Dam	
			Maximum Height m	Volume $m^3 \times 10^3$
1. Pontebba	A.-G.	Tagliamento	23,50	1,55
2. Fusino	A.-G.	Adda	59,00	42,00
3. Furlo	A.-G.	Metauro	62,22	-
4. Pieve di Cadore	A.-G.	Piave	112,00	377,00
5. Cecita	A.-G.	Crati	55,00	60,00
6. Val d'Auna	A.-G.	Talvera (Adige)	53,00	54,00
7. Prà da Stua	A.-G.	Aviana - Sorne (Adige)	42,87	30,50
8. Publino	A.-G.	Adda Alpina	42,00	34,00
9. Forte Buso (Travignolo)	A.-G.	Avisio	110,00	260,00
10. Lago di Valsoera	A.-G.	Orco (Po)	54,00	37,00
11. Beauregard	A.-G.	Dora Baltea	132,00	430,00
12. Piastra	A.-G.	Piastrone-Chienti	87,00	160,00
13. Pian Telessio	A.-G.	Rio Piantonetto-Orco - Po	80,00	380,00
14. Piaganini	A.-G.	Vomano	45,50	26,00
15. Monte su Rei	A.-G.	R.Mulargia-Flumendosa	100,00	-
16. Bellicai	A.-G.	R.Bellicai-R.Corongiu	30,00	-
17. Cancano II	A.-G.	Adda (1° f.	132,00	490,00
		Adda (2° f.	172,00	586,00

Reservoir		Year built	Bodies to which the Dam belongs	
Storage Capacity $m^3 \times 10^6$	Max.Level m.a.s.l.			
0,01	616,00	1901-1902	Friulana di El.	
0,20	1.153,50	1920-1924	A.E.M. - Milano	
1,800	174,28	1921-1952	Unes	
68,50	683,50	1946-1949	Sade	
108,22	1.142,25	1949-1951	Meridionale	
0,412	916,00	1950-1951	Trentina di El.	
1,50	1.041,50	1950-1951	S.A. El.Valeggio sul Mincio	
5,00	2.134,40	1950-1951	A.F.L. Falck	
32,10	1.458,00	1950-1952	Smirrel	
8,40	2.412,00	1950-1953	A.E.M. - Torino	
72,00	1.770,00	1951	SIP	Under construction
20,400	640,00	1955	Unes	
24,000	1.917,00	1955	A.E.M. - Torino	
0,550	400,00	1955	Terni	
304,00	-		Ente Autonom.Flumendosa	Under construction
1,02			Soc. Monteponi	
115,00 240,00	1.898,00 1.938,00		A.E.M. - Milano	Under construction
				LEGEND : A.-G.' = Arch--gravity dams.

to the height of the layer in the case of arch dams with considerable overhang.

As I said at the beginning, the greater skill necessary for the concrete for the more refined structures is no great handicap in Italy, since there is a high standard of skill in our country and many contractors who have specialised in the construction of dams have reached a high level in the utilization of labor. The same is true for the layout; many of our technicians are rather clever in this work.

Of great help to the smooth running of the concrete placement and to its constant good quality are the preliminary studies of mix design which in Italy are generally carried out in the specialised and well-equipped laboratories of University Foundations and of private concerns.

Now with regard to the cost. As to the single elements of the cost of concrete in itself (i.e. apart from the influence of the cost of installations), undoubtedly some of them are a little higher for arch dams than for gravity dams if only for the fact that there must be a higher unit strength. However, this extra cost reduces itself in practice to the higher percentage of cement, which does not normally exceed a plus-difference of 50 kg, per cubic meter and some slight extra cost for formwork. There is another almost negligible difference due to the heavy vibration rendered inevitable by the Italian practice of reducing the water-cement ratio to as low a value as possible. With regard to concrete mixtures I would like to draw attention here to what I also pointed out at the recent Icold Paris conference: Italian mixtures of pozzolanic cement (now commonly used in nearly all dams) include also the pozzolana calculated in the weight as cement, so that the values involved are not comparable with analogous American values. For example, the mass concrete used in the construction of Hungry Horse Dam was made up in the proportion of 111 kg of cement and 56 of fly ash, total 167 kg, as against 200 kg of the ferrico-pozzolanic cement used in the Pieve di Cadore dam.

It must be born in mind that in Italy dams are built very often in high mountain areas or at any rate in very cold regions where the problem of freezing is of first importance. The consequence of this is that even for a gravity dam there is need for special mixtures of concrete and special care in pouring, at least in the neighbourhood of the faces.

Some difficulties in the construction of arch dams may be due to form, on account of the curvature of the surfaces; and to the fact that the curve of the axis in some cases renders some forms of transport and concrete-laying inappropriate. This last difficulty, however, can easily be overcome by widening the operating range of the cable-way equipment or, as was done in the great Piave dam, in that of Travignolo and others, with the use of movable distribution towers on curved rails which follow the ground plan of the structure. Moreover, it should be remembered that gravity dams more often demand longitudinal joints and artificial cooling of the concrete, both of which are only a few cases necessary for arch dams and less rarely also for arch-gravity dams. Furthermore, the diagram of the pouring as a function of height is often more favorable to good utilization of the installations in the case of curved dams than in that of a gravity dam, on account of the fact that the maximum of surface comes in the middle of the period of construction.

At any rate, to conclude the question of costs of many arch and dome dams recently built in Italy, in comparison with gravity structures, we may state that the difference in cost is, in practice, non existent for arch-gravity dams: in short, it is reduced to the influence of the lower concrete volume in the

incidence of the building installation. In other words, the fundamental factor in the unit cost is the volume of concrete, as already made clear by others (see Biadene, Icold report). The unit cost rises, on the other hand, of course, for thin arch and dome dams, but even for these, fundamentally, in inverse ratio to the volume.

In our country, however, the total cost for the arch solution as against the gravity solution cannot, generally speaking, work out higher, because the saving in volume is generally, in percentage terms, far higher than the percentage increase in the unit cost, so that the total cost shows always an actual and substantial saving.

Some Particular Considerations

I think that the above summary is enough to make clear the conceptions and the development of designing and construction of arch dams in their various types in Italy. The general ideas explained above are naturally applied to the single cases which present themselves in practice with some variations according to the ideas of the designers and builders.

Indeed, while in a certain sense one can speak of uniformity of trends in the field of analysis, one cannot say that any such uniformity exists in particular applications, although a certain tendency to standardisation is discernible.

I would like here to give some explanation of a few principles on which many Italian designers and builders have worked and which, as it appears from my experience, are often rather difficult for foreign technicians to rightly understand.

I think, therefore, that it would be of some use to mention these principles, again making it clear that they do not however, constitute normal Italian practice and still less Italian compulsory regulations: they merely reflect a group of ideas and the fruit or the practical experience of some designers.

1) It is often held that Italian engineers tend to adopt symmetrical structures for arch dams: this affirmation does not perhaps express the matter accurately. It is merely a question of suitability which seems to us rational.

The arch and the dome, even if calculated as being composed of a series of independent elements, must always be considered as continuously curved three-dimensional plates as they actually are.

Assymmetry causes a less uniform distribution of stresses with higher maximum and minimum values, than in the symmetrical structure having the same mean stresses with the maximum values (usually fairly high) proper to arch dams. It may therefore be that an assymmetrical solution, while having a smaller surface development than a symmetrical one, might involve greater volume, because, should the stresses be logically kept within the same limits as in a symmetrical solution, often the thicknesses would be substantially increased in certain parts of the dam.

Moreover, it should not be forgotten that the analysis of an assymmetrical structure is more difficult and more delicate in conception, and therefore the design must depend to a great extent on model tests, unless more conservative stresses are adopted.

In conclusion, it is not a question of taking up an absolute position in favour of symmetry, but of choosing between solutions which may both be technically sound but differ in cost: the choice must be based on economic criteria.

At any rate, we must also bear in mind the fact that this geometrical

symmetry applied to the arch, would remain a mere desire from a functional point of view, as generally the characteristics of the rock and the thermal conditions are not equal on both sides of the location; therefore, the symmetry attained in the shape would not be reflected in the static performance.

2) If the conception of symmetry has a relative importance, we give a greater importance to that of geometrical continuity of the structure. The conception of geometrical continuity, however, must be applied, in my view, not only to the arch itself (whose horizontal and vertical sections must vary gradually, as all the geometrical parameters of the structure must do) but also, at least to some extent, to the supporting surfaces.

Our point of departure is that every more or less abrupt variation in the perimeter of the curved plate gives rise to considerable concentrations in the stresses, with static repercussions also on part of the adjacent areas, resulting in marked static difference in the behavior of the structure.

Moreover, this principle of a relative continuity of the surface of the abutments may cause serious increases in the excavations as well as in the volume of the arch, when there are abrupt local variations of the natural profile. For example, a deep recess in the abutment, should a completely continuous perimeter be sought, would render necessary considerable increases of volume also in the adjoining zones. In such cases we are led to eliminate the discontinuity by means of a sort of plug which fills in the recess mentioned, the plug being separated from the curved plate itself by a construction joint; and the same holds good for all the parts which, owing to the irregularity of the rock surface, do not follow a reasonably continuous line for the surface of the supports. (Fig. 19.)

I may here mention that, on the basis of this idea of the continuity of the abutment surface, many Italian designers have eliminated the stepping of the rock in arch dam abutments, achieving excavation lines as far as possible continuous, without abrupt changes of relief.

3) Further, it is recognised that the perfect encastrement on which the analysis of arches is often based is only a hypothesis which very rarely corresponds to reality. For a perfect encastrement the perimetral part of the plate should be locked in a trench of solid rock whose side walls should encase the structure to a sufficient depth; and further, the part to be encased and the adjacent one should be strongly reinforced. We can otherwise admit that the need to realize the perfect encastrement would arise only when the curve of pressure comes out of the middle third of the abutment.

In any case, we think it is only fair to admit that, in general, the abutment of the structure tends to a simple support. It certainly is so when the plate rests on a plug.

Furthermore, the adoption of the simple support permits greater freedom and ease in carrying out the abutments, which in many cases results in a saving of cost. Considering this fact and given the conception of the continuity of the surface of the supports, owing to which in certain points, where the surface is irregular, the part against the rock is separated from the structure by means of a joint, we have thought that it was logical to extend the joint itself to the whole perimeter: that is, to place the arch or dome structure, designed with a continuous perimetral line, on a kind of distribution saddle or cradle running all along the surface of supports. This saddle is separated from the structure by a construction joint, the "perimetral joint."

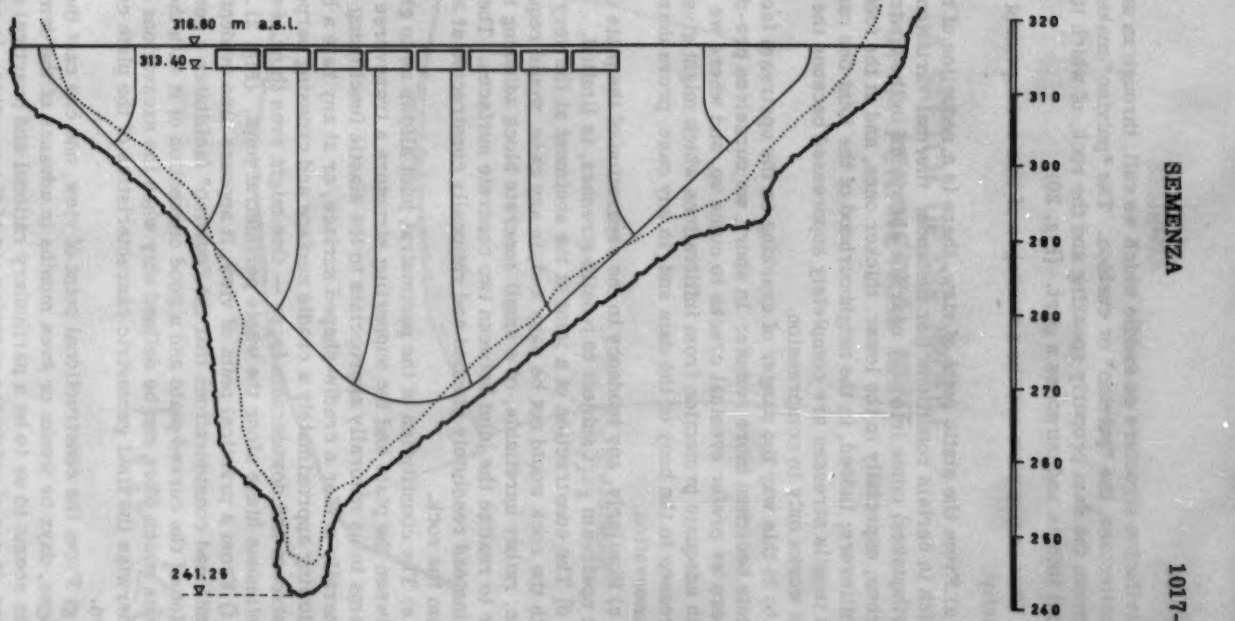
The curved plate thus simply rests on the rock by means of this

PONTE RACLI DAM

ON THE RIVER MEDUNA

DEVELOPED ELEVATION
ALONG THE MIDDLE FIBRE OF THE ARCHES

FROM DOWNSTREAM



distribution structure on saddle which we call, through an analogy from architecture, the "pulvino" or cushion. The "pulvino" makes a clear break between the dam properly speaking and the rock, of which it actually becomes to all intents and purposes a part. (Fig. 20)

4) What in our view are the advantages of this arrangement? I will list them briefly:

a) From the static point of view, there is a reduction of the tensile stresses which in certain conditions (for instance: thermal variations, shrinkage, earthquakes) come into play near the abutments in the upstream face of the arches, especially in the lower, thicker ones, and at the foot of the central cantilevers; indeed, in the neighbourhood of the joint one can consider that the tensile stresses are completely suppressed because the section is free and works only in compression.

b) In this way the danger of cracking on the upstream face near the abutments becomes more remote: in short, we ourselves pre-determine the zone where we prefer eventual cracks to come up, and where we can face them with adequate protection from infiltrations which might give rise to abnormal stresses in the body of the dam and, in any case, prove detrimental to its conservation.

c) Similarly, any tendency to the separation of the plate from the rock in the upstream part subject to tensile stresses, is limited.

d) The construction of a joint at the abutment at the very point of contact with the rock would not be easy and, in any case, would require the construction, rather unreliable, of a small concrete block adhering to the rock in order to realize the joint between two concrete surfaces. The perimetral joint is instead resolutely planned and rationally constructed at a short distance from the rock.

e) The constitution of the perimetral joint allows us to give the surface between the plate and the supporting structure a transverse shape which seems to us naturally appropriate to the elastic functioning of the plate. We generally prefer a cradle-shaped surface, or at any rate a broken surface following approximately a cradle surface and crossing perpendicularly the upstream and downstream faces — One might even think of something like a continuous hinge along the whole perimetral joint. (Fig. 21.)

f) From a practical point of view, it appears also evident from the above mentioned considerations that the "pulvino" facilitates a good geometric definition of the curved plate and a good connection of it with the surface of abutments which often can be defined only when the excavations are complete. Otherwise the final geometric characteristics of the plate could only be fixed then.

g) From the constructional point of view, one can cast the "pulvino" in stages, days or weeks or even months in advance of the dam blocks next to it. This seems to us to be a particulary rational and practical system; furthermore, it can simplify the carrying out of the grouting. In addition, the presence of the "pulvino" facilitates the operations of joining through grouting to the rock.

6) For all the reasons explained in the foregoing paragraphs we have been led to adopt the perimetral joint in several Italian dams.

I already spoke of the original concept and the first application of this device to the Osiglietta dam, owing to the converging ideas of many experts; I

A SKETCH OF THE PROFILE OF A PERIMETRAL JOINT

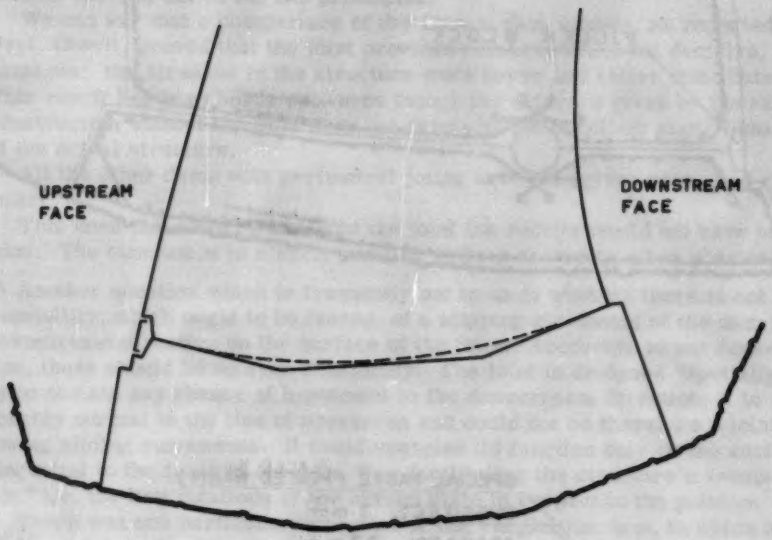


Fig 20

L U M I E I D A M
DETAILS OF THE PERIMETRAL JOINT

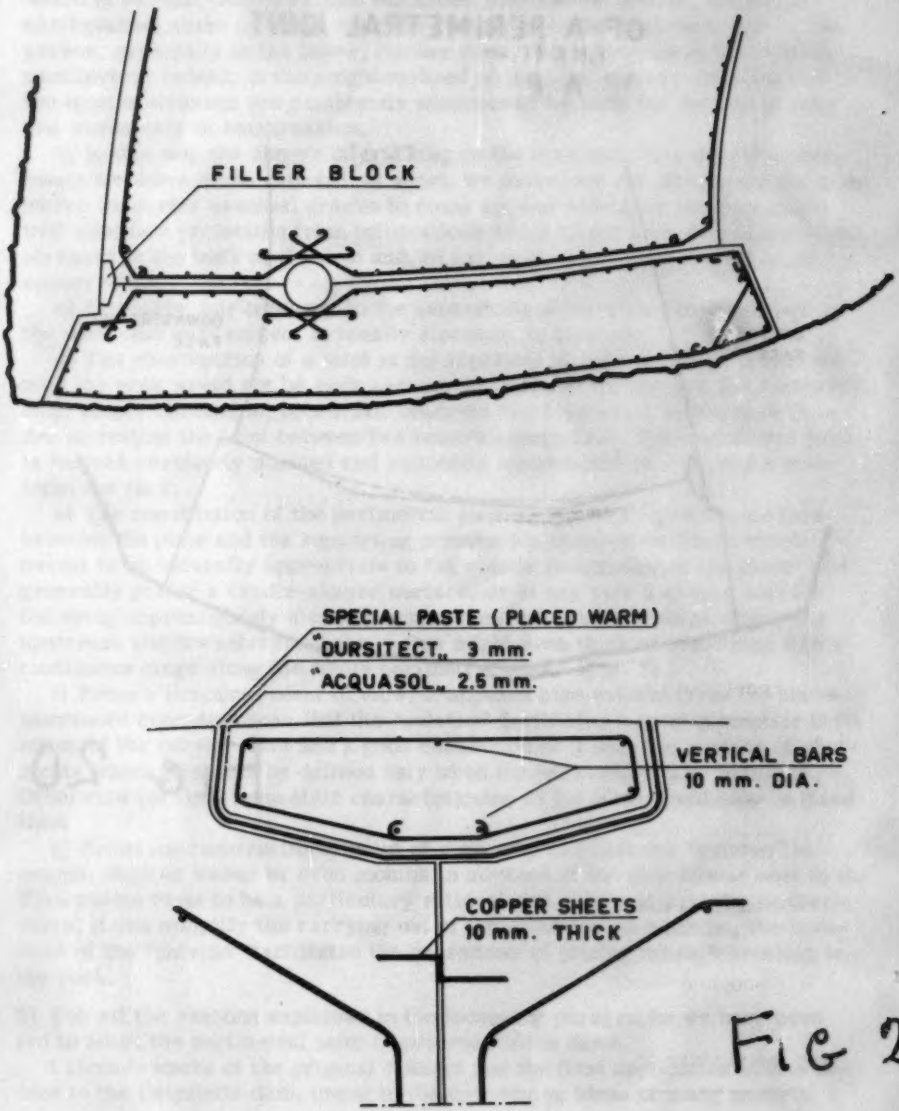


FIG 2

must add that a previous proposal by Mr. Scalabrini for the design of Rocchetta dam had to be abandoned.

After Osiglietta, the joint was applied to many other Italian dams, principally those of the Veneto area and various others among those already placed on record. The most of those in the Vento, built mainly by the Società Adriatica di Elettricità, to which the author belongs, were worked out in collaboration with Prof. Oberti. The others were built by various public bodies and are the work of other designers, including especially Prof. Arredi.

We have been asked many times for news of the practical results of the perimetral joint. Of course, in order to say the last word on these practical results, it would be necessary to build the same dam both with and without the joint so as to be able to compare the behaviour of the two, which is obviously impossible. We therefore had to confine ourselves to comparing the dam models carried out on the two principles.

We can say that a comparison of the Lumiei dam models, as reported by Prof. Oberti, proved that the joint provided remarkable, if not decisive, advantages: the stresses in the structure were lower and better distributed. This result has been borne out—even though the evidence given by the same construction without the joint does not exist—by the excellent static behaviour of the actual structure.

All the other dams with perimetral joints have also given very good results.

This does not mean that without the joint the results would not have been good. The conclusion is clearly positive without excluding other possibilities.

7) Another question which is frequently put to us is whether there is not the possibility, which ought to be feared, of a slipping movement of the dam in a downstream direction on the surface of the joint. According to our conception, there should be no such possibility. The joint is designed "spatially" so as to obviate any chance of movement in the downstream direction; it is roughly normal to the line of pressures and could not be therefore a joint allowing sliding movements. It could exercise its function only on the surfaces tangential to the faces of the dam, thus facilitating the structure's "respiration" i.e. the tiny rotations of the curved plate in respect to the pulvino.

There was one particular case, that of the Val Gallina dam, in which an early model, built without taking into due account the structure weight, had indicated the possibility of sliding downstream; we limited in that case the joint in the base zone of the dam to only the upstream third of the section.

8) I may add that, from the functional point of view, the "pulvino" could be built of concrete with those elastic characteristics which render it able to act as an intermediary between those of the concrete of the arch and those of the rock, so as to increase still further its capacity to work as a transition structure between the arch and the rock: a function already implicit in its conception from the structural and position points of view. One of the functions of the "pulvino" is precisely that of distributing and reducing the stresses in the rock: its form and its dimensions can indeed be adapted (with appropriate widening and re-inforcement) to the characteristics of the rock. Inversely, it is capable of reducing the repercussion on the curved plate of the discontinuities and irregularities in those elastic characteristics along the surface of support.

In particular the "pulvino" can be thickened, widened and, if necessary, strengthened by adequate steel re-inforcements, in order that it can work as

a "bridge" over the weaker areas of the abutment surface. Admittedly, this bridge-function can also be taken over by the adjacent areas of the curved plate itself but the presence of the "pulvino" facilitates this function: the "pulvino" plays a more defined part which can be more easily adjusted and analysed so that the plate confines itself to absorbing a more limited proportion of the abnormal and additional stresses.

9) Another device we use in the arch dams, which is far from being peculiar to our country, but from the frequency of its use can be called a normal conception in Italian technique is that of the discharge of floods over the crest. Our assumption is that for actual safety it is necessary that among the various discharge provisions, at least one ought to be an open spillway followed by an open conduit. Indeed, the consequence of a closed conduit would be a sudden and uncontrollable raising of the water level whenever the peak-flood exceeded the calculated maximum. A crest-spillway probably constitutes in many cases the most economical solution for the open discharge works and one which we strongly recommend. For the above consideration, in many arch and dome dams in Italy the crest has been utilized for the discharge of floods exceeding those which can be normally absorbed by other discharge devices.

Of course—should there not be any special device provided against scour and erosion—the rock must be capable in these cases of taking the full impact of the discharge, although it generally lasts for only a short period. We are actually convinced that, when it is a question of discharging only exceptional peak flows and where the foundation rock is strong enough, it is in some cases not even necessary to arrange for special downstream protection.

I already mentioned the old Cismon dam and the Sottosella dam (1939). There are several more modern examples. In some cases, where a discharge exceeds the calculated maximum, the excess flow has also been allowed to spill over a crest not expressly designed for that function (for instance, Pieve di Cadore).

All these ideas which I have brought to your knowledge are naturally open to discussion. I thought it my duty to submit them to the most full and frank discussion of our eminent American colleagues: I can only say that our practical experience to date is completely successful.

The author wishes to express acknowledgment and appreciation to Professors. Oberti and Arredi, to Messrs. Sensidoni and Scalabrini, and to his staff, whose important additions and suggestions have been invaluable in the drawing up of this paper.

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ARCH DAMS: DESIGN OF THE KAMISHIIBA ARCH DAM

C. C. Bonin,[1] A.M. ASCE, and H. W. Stuber,[2] A.M. ASCE

FOREWORD

This paper is one of a group to be presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee,

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all.

At this time, it is not known exactly how many papers will be printed from the Symposium. So far, twelve papers have been approved: "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993): "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dams," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); and "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018).

As other papers are approved, they will be published in the Proceedings. The interested reader should watch for these papers in following issues of the Journal of the Power Division.

Note: Discussion open until November 1, 1956. Paper 1018 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 3, June, 1956.

1. Eng. Manager, Ebasco Services, Inc., New York, N. Y.
2. Civ. Engr., Ebasco Services Inc., New York, N. Y.

SYNOPSIS

This paper outlines the design of the first arch dam to be built in Japan. Discussed in detail are geologic conditions at the site, basic assumptions for the design, design criteria, and the allowable stresses used. Also included is a brief discussion of the spillway adopted for this project.

INTRODUCTION

On June 25, 1955, the 90 000 kilowatt capacity Kamishiiba Hydro Electric Development was formally dedicated by the Kyushu Electric Power Company of Japan. This development includes the first arch dam in the Far East and is located on the upper reaches of the Mimi River in the central portion of Kyushu, the southernmost of Japan's four principal islands. Figure 1 is an aerial photograph of the dam. The area is mountainous with ranges up to 1500 meters in elevation cut by deep, steep sides, V-shaped valleys. Forests and dense vegetation cover the entire area. Annual rainfall averages approximately 2 500 mm of which more than one-half occurs during May, June and July. October through January is the dry season during which about one-tenth of the rainfall occurs. Typhoons with torrential rains causing short duration high crested floods in the rivers of the area generally occur in August and September of some years. The average monthly flow in the Mimi River at Kamishiiba varies from about 8 cubic meters per second in mid-winter to about 40 cubic meters per second in mid-summer. Five hydro-electric developments utilizing gravity dams with little or no seasonal storage were previously built on the Mimi River below Kamishiiba. The smallest of these has been by-passed and eliminated by the Kamishiiba Hydro Electric Development.

Kamishiiba Hydro Electric Development

Just prior to World War II, Hassoden (The Japan Electric Generating and Transmission Company) began site investigations and power studies for a development with a 90 meter high gravity dam at Kamishiiba. When it became possible to resume these studies after the war, a 110 meter high gravity dam was considered in an attempt to make the development economically feasible. However, the value of the additional energy made available was offset by the cost of the added mass of concrete in the larger gravity dam, and the development did not appear too attractive.

In 1950, Hassoden retained the American firm Overseas Consultants, Incorporated, to review the engineering practices then being used in the Japanese electric power industry. They felt that their engineering practices had not kept pace with the rest of the world during their self-imposed period of semi-isolation and World War II. While assigned to this work, a group of American engineers visited Kamishiiba and suggested that an arch dam be considered for that site. This prompted the Japanese engineers to form a committee to make a comprehensive study of arch dam design and construction. Up to this time they had, in general, rejected the idea of building arch dams in Japan, owing to apprehension of the ability of such thin structures to withstand the frequent and sometimes severe earthquakes occurring throughout the country.

Early in 1952, Overseas Consultants, Inc., was retained by the newly formed Kyushu Electric Power Company to investigate, analyse and study the height and type of dam to be utilized, and to make recommendations on other engineering matters all related to the Kamishiiba Hydro Electric Development. Five alternative heights of dam were analyzed for both arch and gravity types; various locations were considered. Thorough consideration was given to the geology of the site. Aggregate, cement, formwork, manpower, equipment, layout, type of spillway and earthquakes during and after construction were additional factors considered in the study of the problem. As a result, it was recommended that a 110 meter high constant-angle type arch dam be built at Kamishiiba. A significant factor in the choice of an arch dam over a gravity dam was the fact that one year's time would be saved in construction—very important in a power-short country.

At the damsite the Mimi River has cut through a series of palaoezoic sediments composed of very indurated sandstone and slates. These sediments have a general N 70° E strike and dip steeply to the north, and are part of a palaeozoic zone which spreads NNE to SSW across central Kyushu. At the damsite a very thick zone of graywacke covers nearly the entire arch dam foundation, slate being encountered only on the upper 25 meters of the left abutment. However, the graywacke is interspersed by sparsely located thin layers of slates. These were not considered detrimental. The boundary plane between the graywacke and slate has been faulted, the slate along the fault plane being shattered, crushed and considerably weathered. This zone of crushed slate is interspersed with layers of unsound mylonitic rock, graphite and clay. The height of the dam was limited to 110 meters largely because of this zone of unsound crushed rock. This material was excavated and replaced by a concrete mass to form a thrust block approximately 25 meters high. The recommended location of the dam was established primarily by the geology of the site, secondarily by the surface topography.

The Arch Dam

General Description

In August 1952, Overseas Consultants Inc., was retained by the Kyushu Electric Power Company to supervise the design and construction of the Kamishiiba arch dam. At this time work was begun on the final design for the dam.

Kamishiiba Arch Dam is a constant angle type arch dam with a design base circle radius of 143.50 meters and a top central angle of 119° 47'. It has a design arc length of 300 meters measured along the base circle between the right rock abutment and the concrete thrust block on the left abutment at Elevation 483 meters, the top of the dam. The final actual length is 308 meters. The dam is shown in relation to the construction plant area on Plate No. 1. The maximum design height of the dam was 110 meters assuming Elevation 373.0 as the base of the dam. (The lowest point of the actual base was found to be Elevation 369.69, thus making the actual height 113.31 meters). It has a virtually constant central angle between Elevations 483, and 420, varying by no more than five degrees from 116°. Below Elevation 420 the central angle diminishes progressively as shown in Plate No. 2. The principal dimensions and arch data listed are given in meters: t and r

represent the uniform thickness of the arch lamination and the centerline radius of the lamination, respectively. The dam was constructed in blocks 20 meters wide, along the base circle, with the radial construction joints grouted in order to provide as nearly a monolithic structure as possible.

The following tabulation gives the principal water surface levels and other pertinent data:

Maximum design flood with wind and seismic waves	Elev 483.0 m
Normal high reservoir level	Elev 480.0 m
Minimum reservoir level	Elev 435.0 m
Drawdown	45.0 m
Maximum tailwater	Elev 398.0 m
Minimum and normal tailwater	Elev 373.0 m
Maximum silt	Elev 420.0 m
Base of dam (assumed)	Elev 373.0 m

The main body of the dam was founded on the hard compact graywacke at the damsite. However, the foundation rock of the upper left abutment was such that a 25 meter high 30 meter long thrust block was required. See Plate No. 6. At the junction of the arch and the thrust block a concrete gravity cut-off extending to the north to sound rock was adopted. The graywacke on the upper one-third of the right abutment was found to be more closely jointed and deeply weathered than expected, requiring extensive excavation. It was estimated that 225 000 cubic meters of overburden and rock were to be excavated for the dam; actual volume removed was 268 000 cubic meters.

The arch dam contains 323 000 cubic meters of concrete, the thrust block and gravity cut-off 30 000 cubic meters, the spillway chutes and walls 28 000 cubic meters. The dam impounds a reservoir with a total storage of 91 500 000 cubic meters, of which 76 000 000 cubic meters is usable for power generation.

Basic Assumptions

For the design of Kamishiiba Arch Dam it was assumed that:

1) The foundation rock at the site would be homogeneous and strong enough to carry the applied loads with stresses well below the elastic limit.

2) The concrete in the dam would be homogeneous and strong enough to carry the applied loads with stresses well below the elastic limit; Hooke's law applied.

3) The dam would be thoroughly embedded in the rock so that the arches and cantilevers may be considered as integral with the foundation.

4) The vertical construction joints would be grouted before the water load is applied so that the dam may be considered to act as a monolith.

5) Effects of creep have been adequately allowed for by using a somewhat smaller value of the modulus of elasticity than would otherwise be adopted.

In addition to these basic assumptions, more specific assumptions for analyzing arch and cantilever elements will be presented later.

Loads

A clear line of demarcation should be drawn between "external loads" and "elastic deformations," although they are intimately related. "External loads" are forces or influences acting on the outside surfaces of the dam. "Foundation deformation" may be considered to satisfy this definition; however, it has been found preferable to list this under "elastic deformations" to be discussed later. Temperature drop which is an "elastic deformation" has been considered to be an "external load." Its counterpart "shrinkage" is an "internal influence"; however, it is difficult to separate the two, and it is not necessary to do so when the vertical joints of the dam are grouted after the concrete has cooled to or below its mean annual temperature and the effect of further shrinkage may be neglected. Therefore, negligible residual shrinkage is considered to be included in temperature drop for the individual laminations in this design. The active "external loads" that were considered are static water pressure, silt pressure, inertia of masonry during earthquake, dynamic water pressure during earthquake, weight of dam, uplift pressure, weight of water on surface of dam and temperature variation from the mean annual value.

Horizontal loads are resisted by both arch and cantilever elements. They were apportioned by trial until coincidence of deflections of both elements was obtained. Formulas used for obtaining total unit loads are:

Static water pressure = $w_1 x$	(tons / M^2)
Silt pressure = $C_e W_3 X^1 = 0.66 X^1$	(tons / M^2)
Inertia of masonry = $K_1 W_c t \cos \theta$	(tons / M^2)
Inertia of water = $7/8\ w_1 k_1 hx \cos \theta$	(tons / M^2)

in which

w_1 is the specific gravity of water	(1.0)
w_2 is the specific gravity of sand in air	(1.8)
w_3 is the specific gravity of sand in water	(1.1)
w_c is the specific gravity of concrete	(2.3)

C_e is a constant = 0.6

x is the depth of water from the surface elevation to the elevation of the point under consideration

h is the maximum depth of water behind the dam

K_1 is the earthquake acceleration = 0.12 reservoir full
= 0.06 reservoir empty

x^1 is the depth of silt measured from Elevation 420 to elevation of point under consideration

t is thickness of arch lamination

θ is the angle between the assumed direction of earthquake motion and a normal to the face of the dam at the point under consideration.

For design purposes the elevation of the water surface, reservoir full, was taken to be the same as the top of the dam or Elevation 483.0 because of the effects of abnormal floods, wind and seismic waves. The formula for

silt pressure was taken from the publication, "Design Standard for High Dams for Hydro-Electric Developments" by the Natural Resources Board of Japan. In the factor $C_e w_3 x^1$;

$$w_3 = w_2 + w_1 + vw_1$$

where v equals the percentage of voids, here taken as 30%.

Earthquake forces were assumed acting in a downstream direction with the reservoir full thereby increasing the stresses due to water load and temperature drop. The condition of reservoir full with earthquake forces acting across the valley normal to the thrust block was also investigated. For reservoir at low water level the earthquake forces were assumed to act in upstream direction.

Vertical loads act on the cantilever elements only. The cantilever elements are bounded by radial lines one meter apart on a circumferential plane passing through the center line of the Elevation 483 lamination. The actual weight of the element was computed assuming the specific gravity of the concrete as being 2.3. The weight of the water on the cantilevers was determined at each elevation under consideration and used as a vertical load. Uplift pressure has been divided into two parts; namely, that which acts on the base of the dam at any elevation and that which acts on the overhanging portion of the dam between the heel and the furthermost projection upstream of the heel of the dam. The uplift pressure on the base of any cantilever was computed on the basis of 35% of the pressure at the heel of the dam and varying uniformly to zero at the toe of the dam. Where the tensile stress of the cantilever was higher than the allowable stress a crack was assumed and the uplift over the cracked area was computed for 100% of the head and held constant over the cracked area of the concrete. Over the uncracked area the uplift varied from 35% to zero.

Temperature deflections of the arch elements were added to the water load deflections before adjusting the arch and cantilever movements. The temperature load was applied directly to the arch elements only and not directly to the cantilever elements since temperature changes in vertical sections cause negligible horizontal movements. Increases in temperature in the arch elements work against the water load and were neglected except for the case of minimum water level. Decreases in temperature in the arch elements work with the load and were included in the analysis. The maximum probable decrease in temperature below the temperature existing at the time of grouting the joints is the change which was included in the stress analysis. The vertical joints were to be grouted after the concrete had cooled to the mean ambient temperature of 15.6° C.

Temperature changes vary with the thickness of the arch and were computed from the equation

$$F = 340 / t + 8$$

in which F is given in degrees Fahrenheit and t is the thickness of the arch ring in feet. They were assumed constant over an entire arch element. To prevent the formation of cracks in the dam to the greatest possible extent it was necessary to limit the temperature rise of the concrete as much as possible. To accomplish this, a concrete mix with minimum amounts of moderate heat type cement was used. Air entraining admixture was added to improve the workability of such lean concrete mixes. Other important

considerations were the rate of placing concrete and the height of the lifts. Concrete lifts that were in contact with foundation rock were limited to one meter, elsewhere two meter lifts were specified. Before grouting contraction joints the concrete was to be cooled to mean ambient temperature by circulating refrigerated water through systems of cooling coils embedded in the concrete lifts.

"Elastic deformations" were included in analyzing the stresses of Kamishiiba Arch Dam. However, certain elastic deformations counteracting water load stresses have been omitted. Among these are swelling of concrete and temperature rise. Present knowledge indicates that they would be beneficial in most cases, but they have been included only in the case of low water where they would, theoretically, be detrimental. The elastic deformations that were included in the design are:

Moment deformations
Thrust deformations
Shear deformations
Foundation deformations

The external loads shorten the arches and introduce thrusts, shears and moments. Stresses so produced are known as rib shortening stresses, radial shear stresses and bending fiber stresses respectively. The effect of Poisson's Ratio was ignored except when evaluating the shear modulus of elasticity. All the effects of the "elastic deformations" mentioned have been included in computing the stresses for the design cases considered. The following physical constants were used.

Temperature coefficient		0.000010 /°C
Modulus of elasticity of concrete		E_c = 200 000 kg/cm^2
Modulus of elasticity of hard sandstone		E_{R1} = 250 000 kg/cm^2
Modulus of elasticity of slate		E_{R2} = 200 000 kg/cm^2
Poisons Ratio, concrete		0.20
Poisons Ratio, rock		0.20
Shear Factor	k = 1.25 radial	1.00 tangential
Shear modulus ratio	n = 3.00 radial	2.40 tangential

Design Criteria

The design criteria described here establish conditions for which complete trial load analysis were made for the completed dam and the conditions for which the stability of the dam was investigated while it was under construction.

To analyze stability during construction the loads considered as acting on the structure, or portions of it, were: weight of masonry and earthquake acting upstream. One of the objects of this study was to determine the height to which each individual block could be built without benefit of the support of adjacent blocks, and to determine the magnitude and variation of stresses at each corner of the base of each block under consideration as the height varied from the foundation to the top of the dam at Elevation 483. Blocks 11-12-13 on the right abutment were considered to be the most critical. Each of these was considered individually, and they were also investigated as a unit.

The blocks were considered safe against sliding and overturning when the following conditions were satisfied.

1) Coefficient of friction equal to or less than 0.80; i.e.

$$\frac{\text{Summation of horizontal forces}}{\text{Summation of vertical forces}} = 0.80$$

2) Shear-friction factor of safety equal to or greater than 4.0; i.e.

$$S_{s\text{-}f} = \frac{f\,(W) + rs_a\,A}{P} = 4.0$$

In which
- P = Horizontal forces
- W = Vertical forces
- f = Coefficient of friction
- r = Ratio of average to maximum shear on joint
- Sa = Unit shear strength = 200 ton/m^2
- $S_{s\text{-}f}$ = Shear-friction factor of safety
- A = Area of section of base

3) $\frac{\text{Resisting moment}}{\text{Overturning moment}} = 1.2$

Load conditions for which complete or partial trial load analyses were made are:

1) High water Elevation 480
2) High water Elevation 483 + silt + temperature drop
3) High water Elevation 483 + silt + temperature drop + earthquake inertia downstream
4) High water Elevation 483 + silt + temperatuer drop + earthquake across valley normal to thrust block
5) Low water Elevation 435 + temperature rise + earthquake inertia upstream, no silt
6) Stability During Construction, no water, earthquake inertia upstream.

It was not considered necessary to obtain a complete "Convergence of Adjustments" for each case. The extent of the convergence was limited to radial adjustment only for case 5, and radial adjustment and shear adjustment for case 4. For the basic design, case 3, a complete convergence of adjustments was made in the order of R_1; S_1; T_1; R_2; where:

- R = Radial adjustments
- S = Tangential or shear adjustments
- T = Twist adjustments

For Case 4 the order of convergence was S_1; R_1. The subscripts indicate the order of the adjustments. R_1 for instance, means the first completed radial adjustment.

Case 1 was analyzed because this was considered to be the most probable condition for long periods of time and the one most likely to provide stress evaluations for comparison with the values to be obtained in the future from the Carlson measuring instruments embedded in the dam.

A complete trial load analysis was required for Case 3 because this case involved both static and dynamic loads and was the most severe condition of

loading. The results of this analysis are shown in Plate No. 7. Wind and seismic waves were included in the dynamic loads. The height of wave due to earthquake was computed from the formula.

$$h_1 = 0.75 + \frac{F^{1/2}}{3} - \frac{F^{1/4}}{4}$$

in which h_1 is in meters and F is the distance of reach in kilometers. The height of wave due to earthquake was computed from the formula

$$h_2 = \frac{KT}{2} (gH)^{1/2}$$

in which

K = Equivalent horizontal acceleration = 0.12
T = Frequency of earthquake = 0.1 to 0.3
g = acceleration of gravity = 9.8
H = Depth of water = 107 meters

For convenience of computation the effect of the above loads was added to the normal high water level used to evaluate static water pressure on the dam. This resulted in a maximum water level of Elevation 483. Dynamic earthquake forces were computed for water at Elevation 480. Temperature drop is given as a condition for this case because it made the stresses higher. Actually, high water occurs in the hot months when a temperature rise is more probable.

A complete trial load analysis was required for case 2 in order to provide complete date for comparison of conditions with and without earthquake. This gave a clear picture of the effect of earthquake on Kamishiiba Arch Dam.

Allowable Stresses

It was proposed that the following 91 day concrete strengths be used as a basis for establishing the allowable design stresses:

W/C	91 day compressive strength, Kg/cm^2
.55	265
.50	320
.45	365

It was also proposed that the minimum value of 265 kg/cm^2 could be increased by 10% by the use of an enriched concrete mix where necessary. Therefore, a minimum available compressive strength of 291.5 kg/cm^2 could be assumed. For a Safety Factor of 5 the allowable stresses for static loads and temperature would be:

Final compressive principal stresses
normal mix 53.0 kg/cm^2
enriched mix 58.3 kg/cm^2

Final tensile principal stresses should not exceed 15% of the allowable compressive stresses, or for
normal mix 8.0 kg/cm^2
enriched mix 8.7 kg/cm^2

Concrete production for Kamishiiba Arch Dam was to be under constant quality control, thus, easily meeting this minimum 91 day strength. However, in accordance with Japanese code requirements the allowable stresses were reduced by 10 % to allow for possible variations. The final maximum principal stresses for a combination of static load and temperature therefore did not exceed the following values:

	normal mix	enriched mix
Max. compression	47.7 kg/cm^2	52.5 kg/cm^2
Max. tension	7.2 kg/cm^2	7.9 kg/cm^2

and these values were increased by 15% for maximum allowable principal stresses under earthquake conditions.

The final layout of the dam was the result of many trials. Each lamination was considered as a unit, then was inspected in relationship to the entire dam. Fowler's curves were used as a guide to the general adequacy of each arch ring in the preliminary studies. These early stress explorations were regarded as preliminary design. After a satisfactory layout was selected, a more exact determination of the probable load and the induced stresses at the various elevations was made by the procedure outlined in pamphlet R/C 21 of the Portland Cement Association. This determination was basically a design procedure and served as the preliminary step for radial adjustments in the trial load analysis. The calculations required in this procedure were tabulated for ten evenly spaced elevations. In the design, and subsequently in the analysis, only the portion of the dam between the base and elevation 472 was considered effective as an arch.

Spillway

The spillway consists of four bays controlled by Tainter gates 9.0 meters wide and 8.3 meters high, in pairs near each end of the dam. General arrangement is shown in Plate No. 1, and details can be seen in Plates No. 2, 4 and 5. Spillway is capable of discharging 2 160 cubic meters per second when reservoir rises to Elevation 481.75 (1.75 meters above the maximum normal operating level).

In addition to the arrangement adopted, two alternative plans were studied in detail; namely, a free overflow spillway located in the center of the arch, and a tunnel spillway through the left abutment. For economic and other reasons neither of these plans was as attractive as the two chute spillways adopted.

Extensive model studies were made at the Electric Power Central Research Laboratory, Tokyo. Careful analysis was made of the approach conditions to the spillways which resulted in the layout shown in Plate No. 5. The orientation of the chutes was model tested very carefully for the purpose of having the two chute jets impinge upon each other, thereby obtaining effective energy dissipation of the overflow. The details of the chutes as shown on Plates No. 2 and No. 4 were the result of these model studies.

Since the completion of the project, the spillways have experienced large rates of overflow. Operation has proven very satisfactory, confirming the behavior predicted by the model tests. Two photographs showing the spillway in operation are shown in Figure 2. Discharge is 680 cubic meters per second in each chute.

ACKNOWLEDGMENT

To the Japanese engineers of the Kyushu Electric Power Company who worked so hard and diligently in carrying out the laborious computations required by the Trial-Load Method, the authors express their appreciation. Also, acknowledged is the invaluable help of two American engineers, A. L. Parme and M. F. Fornerod, who joined the authors in Japan to help with the training and work with the Japanese engineers.

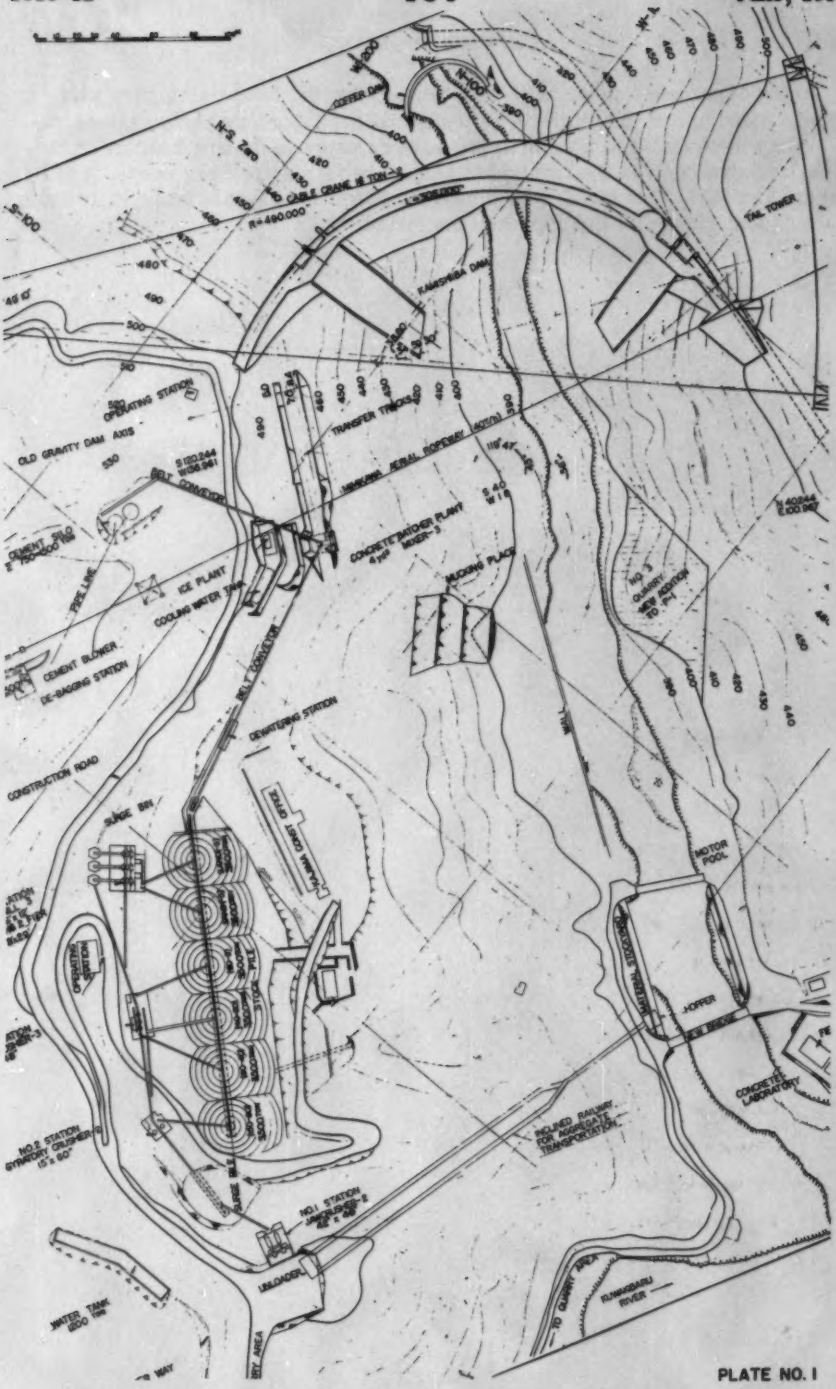
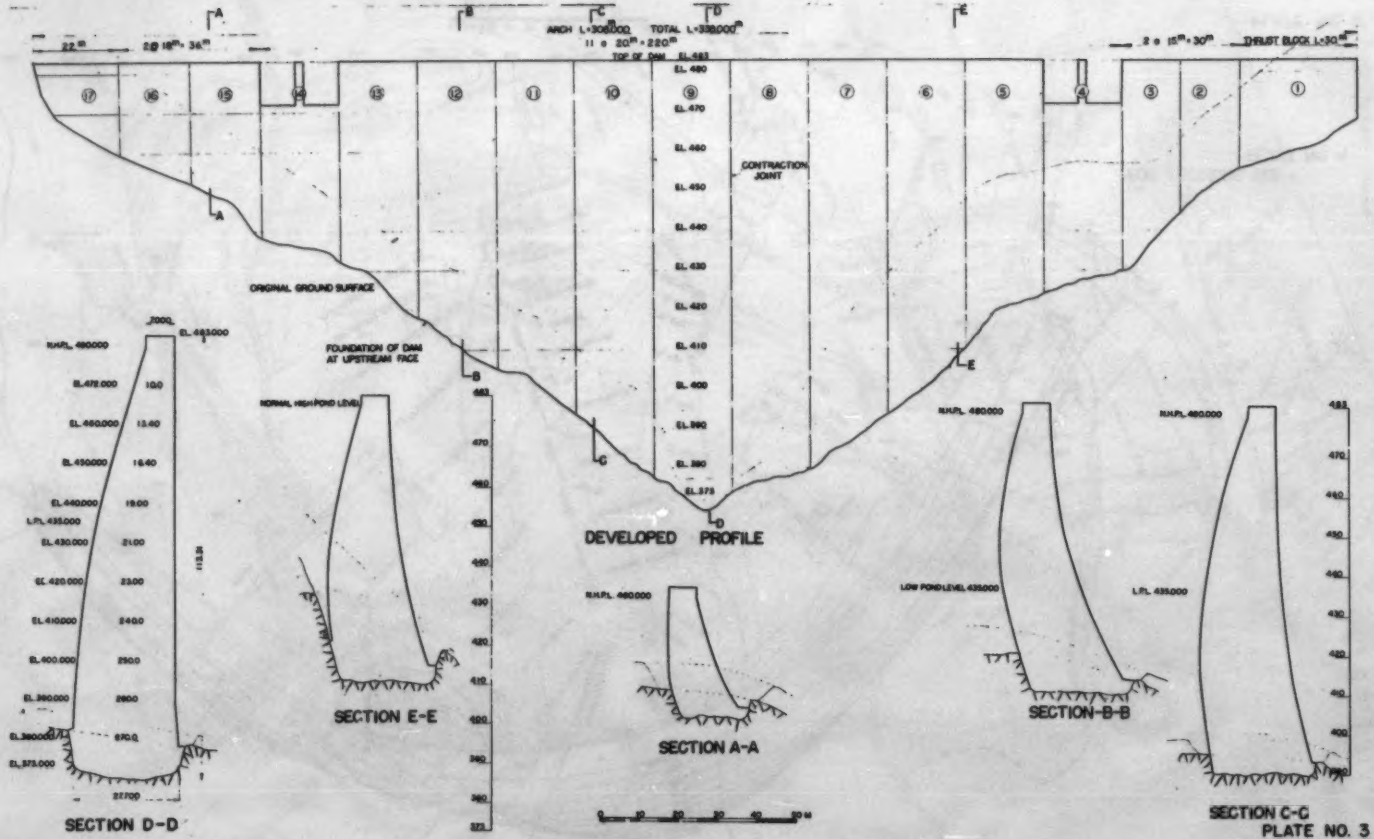

COFFER DAM
N-100
N-200
N-S Zero
MID CABLE CRANE 18 TON
R=490.000
L=300.000
TAIL TOWER
HAMBURA DAM
TRANSFER TRACKS
OLD GRAVITY DAM AXIS
OPERATING STATION
HAMBURA AERIAL ROPEWAY
BELT CONVEYOR
CONCRETE BATCHER PLANT
MIXING PLACE
ICE PLANT
COOLING WATER TANK
PIPELINE
CEMENT SILO
CEMENT BUNKER
DE-BAGGING STATION
CONSTRUCTION ROAD
DEWATERING STATION
SURGE BIN
CHAKMA CANAL OUTLET
MOTOR POOL
HOPPER
CONCRETE LABORATORY
HOLMES RAILWAY FOR AGGREGATE TRANSPORTATION
NO.2 STATION GYRATORY CRUSHER
NO.1 STATION JAW CRUSHER
RECLAIMING BELT
UNLOADER
WATER TANK
NO 3 QUARRY NEW ADDITION TO PIT
NO 2 QUARRY
KANGARU RIVER

PLATE NO. I

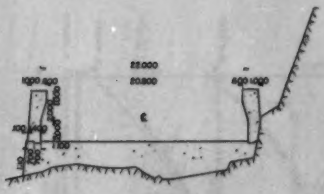
FOR SECTIONS SEE
PLATE NO. 4
PLATE NO. 2



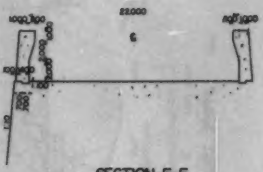
PO 3

June, 1956

PLATE NO. 3



SECTION C-C



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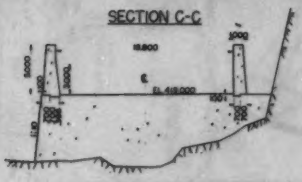

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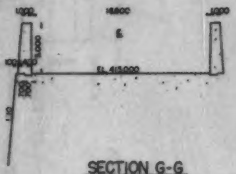

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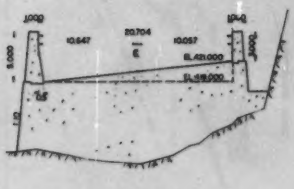
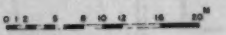
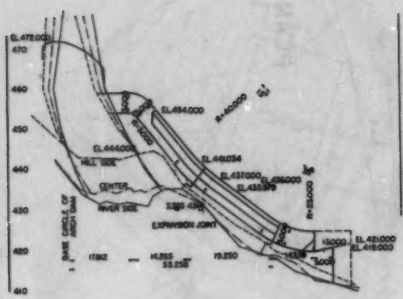


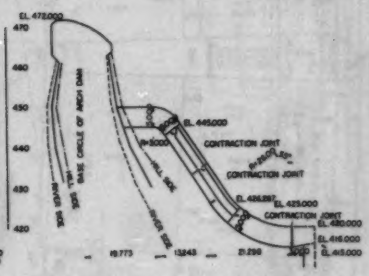
SECTION E-E

SECTION H-H





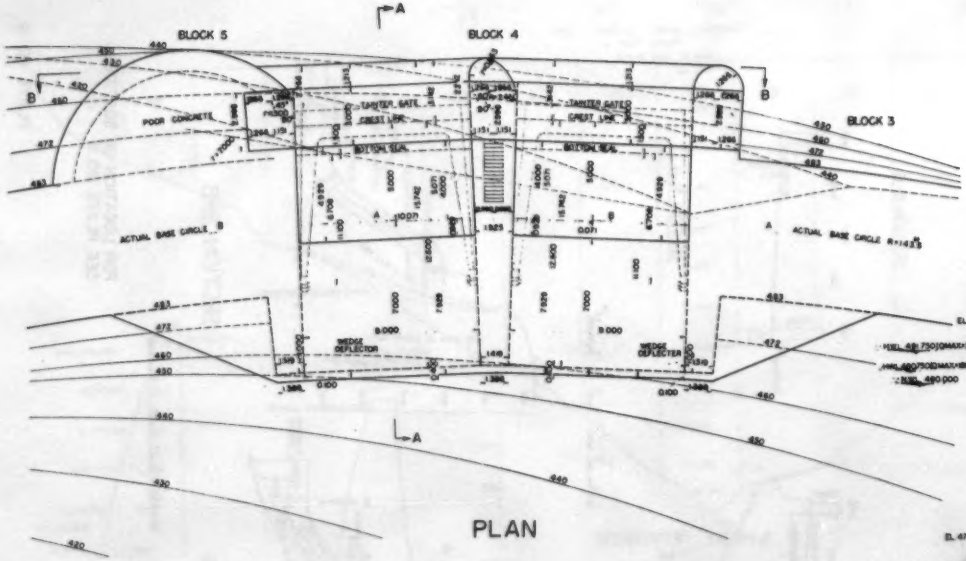
SECTION A-A



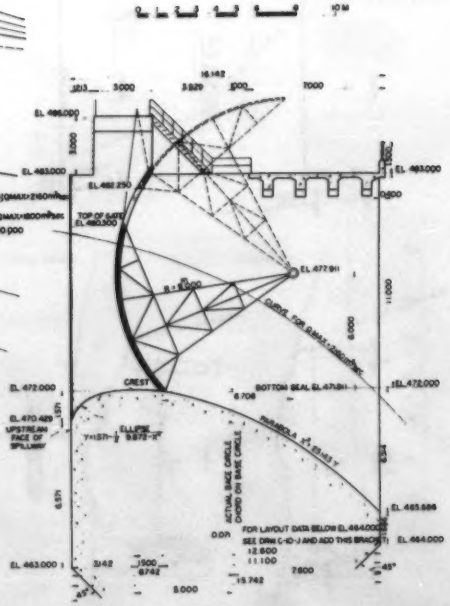
SECTION B-B

FOR LOCATION OF SECTIONS
SEE PLATE NO. 2

PLATE NO. 4



PLAN

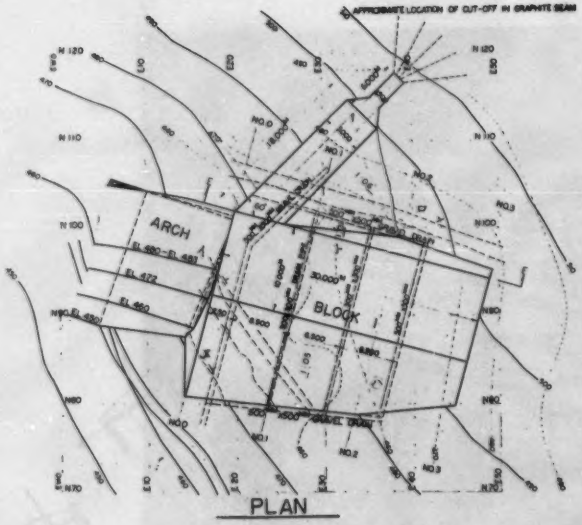


SECTION A-A

PO 3

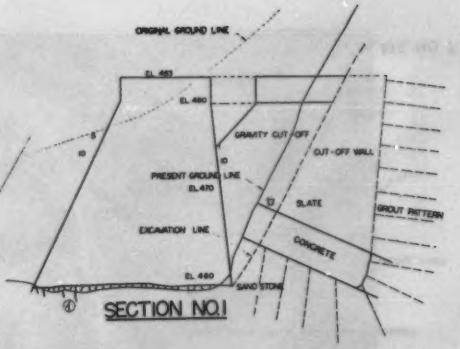
PLATE NO. 5

June, 1956

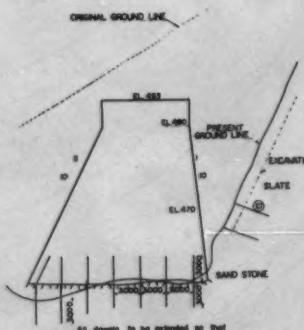


PLAN

SECTION E-E

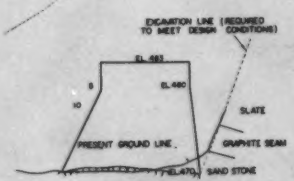


SECTION NO.1



SECTION NO.2

SECTION NO.0



SECTION NO.3

NOTES :

1. 30$^{mm}\phi$ Dowel bars will be embedded over the whole area of the foundation base of the thrust block, and the area of the foundation base of the cut-off which Lies south of the graphite seam.

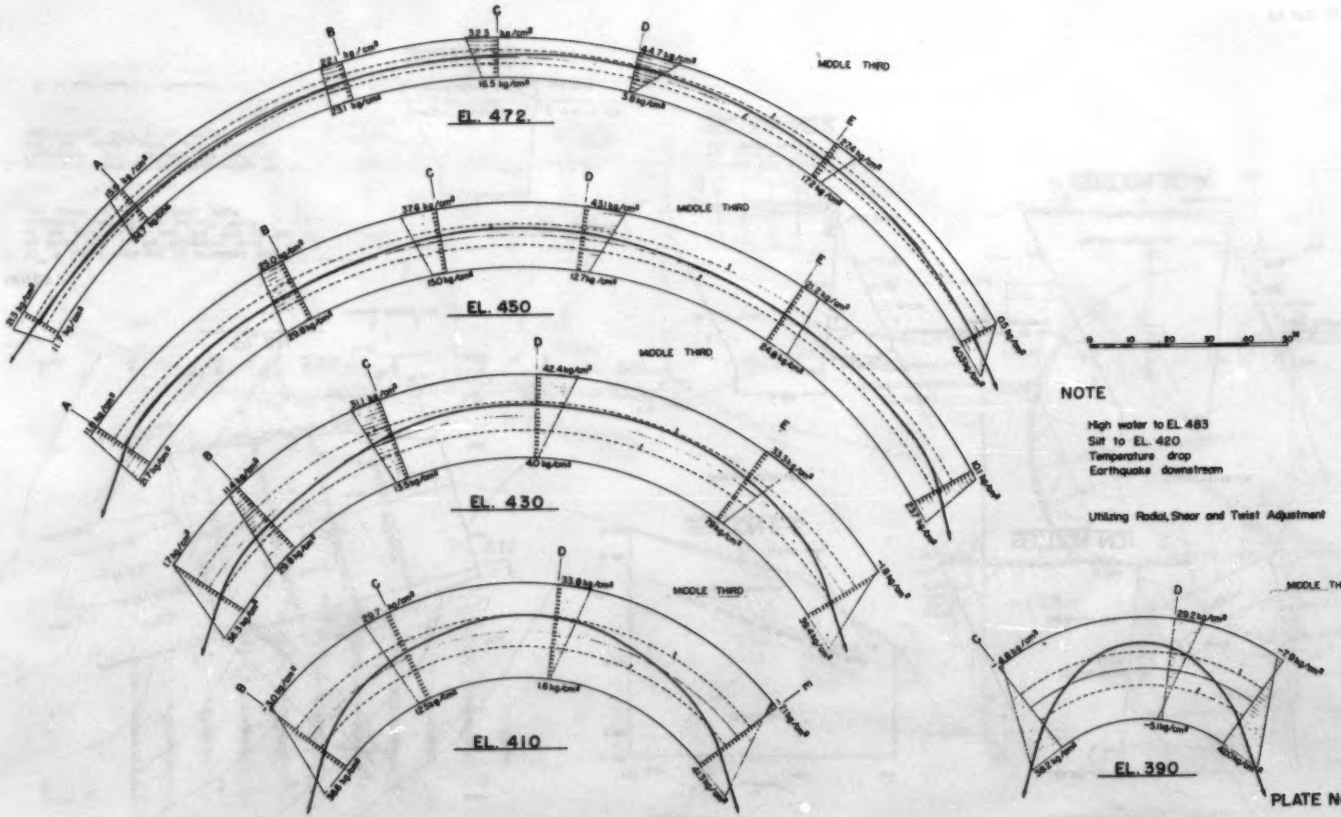
2. Reinforcing bars will be placed along the construction joint between the cut-off and the thrust block with spacing approximately 500 ± on center.

ASCE

BONIN - STUBER

1018-17

PLATE NO. 6



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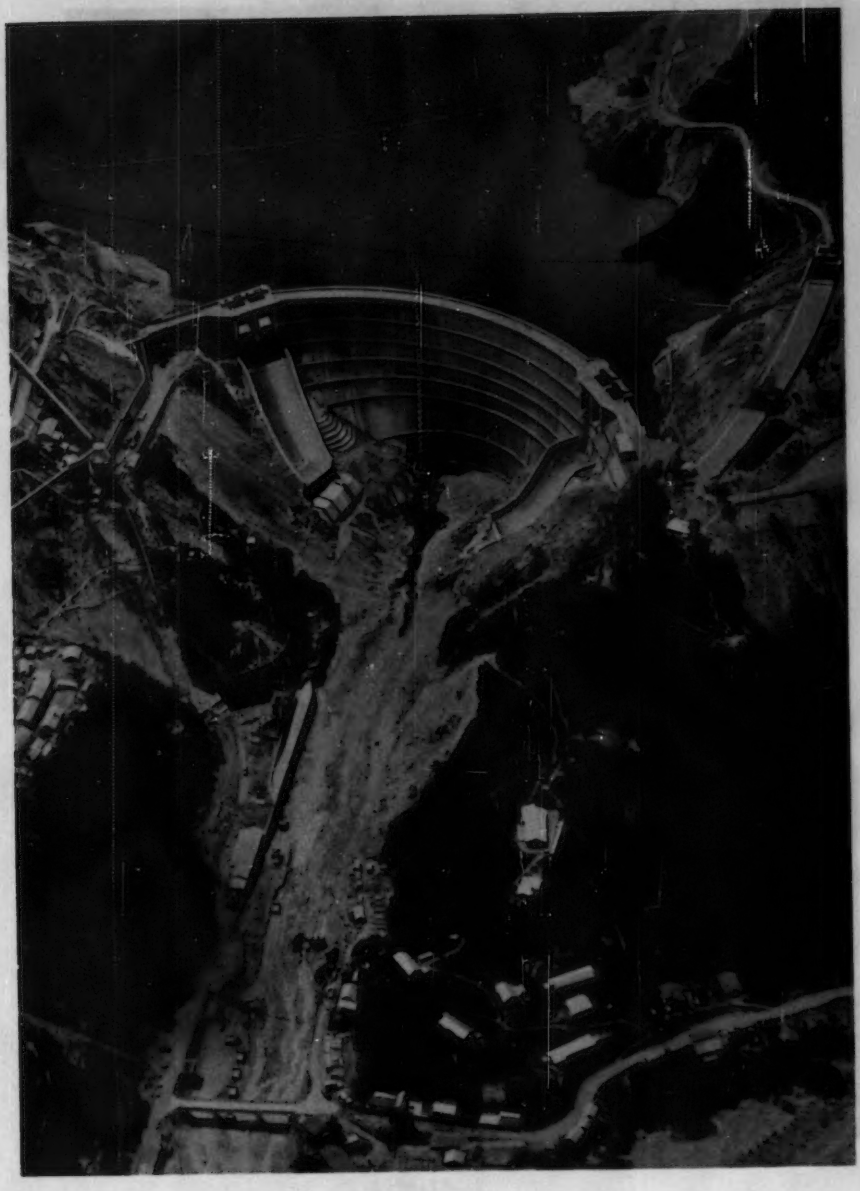
PO 3

June, 1956

PLATE NO. 7

KAMISHIBA DAM

FIGURE 1



KAMISHIIBA DAM

FIGURE 2

680 Cubic Meters per Second per Chute

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The Power Division also has representation on three joint committees of the American Society of Mechanical Engineers; namely, those on Prime Movers, Pumping Machinery and Water Hammer.

The San Francisco Section of the American Society of Civil Engineers has an active Power Division group whose recent activity is reported as follows:

The Power Division of the San Francisco Section of the American Society of Civil Engineers was organized in September of 1953. Since that time it has held five successful and interesting dinner meetings at which speakers presented illustrated lectures on all aspects of power engineering and development. Division members and guests enjoyed talks on "Progress toward Economic Nuclear Power"; "Basic Concepts for Determining Hydroelectric Power Plant Installations"; "Diversified Civil Requirements of Steam Power Plant Design"; and other subjects of interest to all.

The formation of such local groups is to be encouraged.

Coming Meetings

October 1956—Pittsburgh, Pennsylvania—A program of papers on thermal and nuclear power will be presented and a very interesting visit made to the Shippingport Nuclear Power Plant.

February 1957—Jackson, Mississippi—The program for this meeting is being arranged.

June 1957—Buffalo, New York—A Symposium on Pressure Conduits will be presented by the Committee on Progress in Power Plant Design and one or two sessions will be devoted to power developments in the Niagara and St. Lawrence area. An interesting visit to points of interest is planned.

Contributions to the Newsletter

Members of the Power Division are invited to submit to the secretary any contributions of interest for the Newsletter and are also urged to consider contributing to the technical work of the division by preparing papers or discussion. It is pointed out that papers prepared for publication without regard to possible use at a meeting will be especially welcomed. Any of the members of the Executive Committee or of the other committees listed will be glad to hear from possible contributors of technical papers.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

VOLUME 81 (1955)

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

OCTOBER: 809(ST), 810(HW)[c], 811(ST), 812(ST)[c], 813(ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

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FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998 (SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006 (HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

c. Discussion of several papers, grouped by Divisions.

c. Discussion of several papers, grouped by Divisions.

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